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PROCEEDINGS:

The Efficiency of Surface Treatments on the Permeability of Concrete*

BY GEORGE W. WASHA†

INTRODUCTION

THE PERMEABILITY of concrete to water has long been a bothersome problem to the engineer. Many methods of treating concrete to obtain a desired impermeability have been used with varying success. They can all be placed under four groups as follows:

1. Proper proportioning of the concrete mix.
2. The use of integral materials added before the concrete is mixed.
3. The application of surface treatments to the hardened concrete.
4. The use of membranes made of several layers of fabric cemented to the concrete with hot asphalt.

This paper deals entirely with the methods of the third group.

Due to lack of definite knowledge on the subject of permeability of concrete the American Concrete Institute in 1921 appointed a committee to investigate this matter. The outlined program was very comprehensive and sub-committees were appointed, each to investigate a phase of the subject. Prof. M. O. Withey, of the University of Wisconsin, was a member of one of these sub-committees and under his direction the work on surface treatments was begun by C. L. Neumeister and R. S. Phillips who carried out tests in 1924 and 1926. The retests made in 1928 on specimens tested in 1926 were carried out by Prof. P. T. Norton. The tests in 1931 and 1932 were carried out by the author who also assembled the data collected during eight years. All tests were made in the Materials Laboratory at the University of Wisconsin.

Eighteen different compounds were investigated, and the number of individual tests per compound varied from four to fourteen. From the tests the efficiency of the treatments and the flow after treatments were obtained. Retests gave the effect of out-door exposure on the treatments. In addition, the effect of length of curing period on the

*See note, p. 8, as to availability of more complete report of this study.—EDITOR
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efficiency of cement grouts was briefly studied. In all cases, the method of treatment used was one that supposedly would give the best results. With commercial compounds the directions of the manufacturer were followed as closely as possible. The complete program required more than 250 tests.

The ultimate aim of this work was to ascertain the merits of some of the commonly used surface waterproofing compounds, and to produce definite numerical data for this type of treatment.

MATERIALS AND METHODS OF APPLICATION

Materials for surface treatments for concrete may be divided into two main classes: those which penetrate the surface of the concrete filling the pores, thus rendering the surface less pervious to moisture, and those materials which form surface films.

The penetrative materials may be inert and owe their effectiveness to their pore or void filling properties, or they may react with constituents in the treated surface, forming compounds of greater volume and greater pore-filling capacity, and less soluble in water. These materials are also used extensively as hardeners for concrete and to prevent surface dusting and disintegration.

Since this series of tests began in 1924 it was deemed advisable to find out whether any radical changes in the various commercial compounds had taken place since that time. Replies received indicated either no changes or changes of a minor character, so that most compounds on the market now can be assumed to be practically the same as those tested in 1924 and 1926.

The methods of treatment are listed below in classes according to type. The compounds listed in each class include only those actually tested.

Class 1. This group of treatments consists of water solutions of inorganic salts which react chemically with the constituents in the concrete with a subsequent deposition of insoluble material in the pores of the concrete.

a) Fluosilicates, (magnesium and zinc particularly).
b) Sodium silicate.
c) Zinc sulphate.
d) Hard-n-Tyte plus Hard-n-Tyte Filler.

Class 2. Water suspensions of substances or mixtures of substances of a pore-filling character or which react chemically with each other or with constituents in the concrete and form pore-filling compounds.

a) Iron filings and ammonium chloride.

b) Casein in ammonia.

Class 3. Soaps.

a) Water solutions of alkali soaps. (Truscon Por-Seal "B.")

The soaps react with the hydrated lime in the concrete forming insoluble calcium soaps of a water repellent nature.

b) Solutions of alkaline earth or heavy metal soaps in volatile solvents. (Transparent Driwal.)

Upon application to a concrete surface the solvent evaporates depositing the solid matter, in the form of calcium, aluminum, magnesium, zinc, and iron stearates, oleates or resinates, in the pores.

Class 4. Combinations of solutions, in two or more applications, which react chemically with each other in the pores of the concrete, filling them with substances of a water repellent or insoluble nature.

a) Soap solutions and inorganic salt (Sylvester Process).

b) Inorganic salt solutions ($BaCl + Na_2SO_4$).

Class 5. Solutions of Liquid and Solid Hydrocarbons. These materials consist of heavy petroleum distillates such as lubricating oil or paraffin dissolved in volatile solvents such as gasoline. The solid matter is deposited in the pores of the concrete upon evaporation of the solvent.

a) Por-seal "A."

b) Colorless Waterproofing.

c) Colorless Minwax.

Class 6. Bituminous coatings and Membranous Systems. The treatments of this group tend to produce films or membranes over the surface of the concrete.

a) Asphalt emulsion.

b) Asphalt emulsion fibrated with asbestos fibers.

Class 7. Miscellaneous.

a) Neat cement wash.

b) Cement grouts.

c) Cement grout with Sika.

TESTING APPARATUS

The apparatus for measuring the flow of water through the specimens is illustrated in Fig. 2. The specimen is sealed in a casting or holder, as shown, with hot asphalt, and this is bolted to another casting to which the bottom of the permeability tube is attached. Specimens are removed from their holders by heating the asphalt with a coil of high resistance wire. Permeability is measured by filling the system with water, applying air pressure at 40 p. s. i., and noting the subsidence of the water in the gage glass with time.

TESTING PROCEDURE

For testing the effectiveness of surface waterproofings, specimens initially permeable are required. The mixes used in the tests of 1924 and 1926 varied from 1:2¼:5 to 1:3:6, by weight; those in the tests of 1931-32 varied from 1:2:3½ to 1:3½:6½. The first step in the

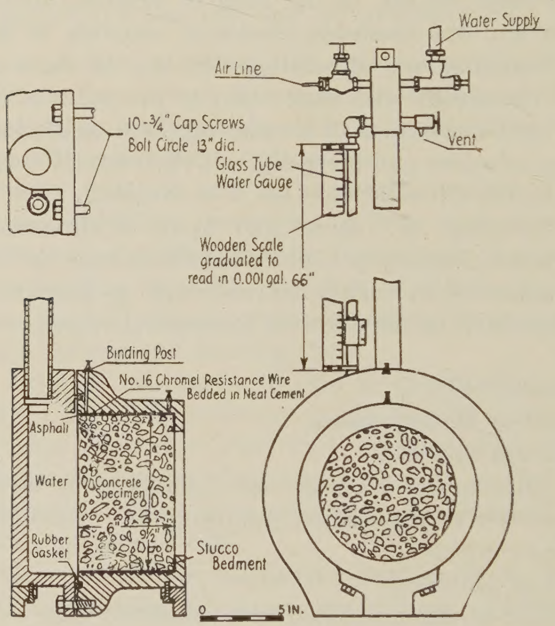

FIG. 2—APPARATUS FOR TESTING THE PERMEABILITY OF CONCRETE TO WATER

assembly was to mount the specimens in the holder in such manner that flow occurred from bottom to top of the specimen as originally cast. The castings were then bolted together, the system filled and air pressure applied. After testing for air and seal-leaks, the test run was made for 50 hours at 40 lbs. pressure. At the end of this period the specimen in its holder was detached from the assembly, and the inner surface of the specimen treated according to detailed instructions—either those given by the manufacturer, or those developed for

the most effective results. After treatment, the apparatus was again assembled and the treated specimens were again tested for 50 hours. Finally the specimens were removed from their holders, and the specimens to be retested were stored in the weather in such manner that all coatings received the same exposure.

COMPUTATIONS

Cumulative leakages in gal. per sq. ft. were plotted against time. Previous tests had indicated that the rate of flow through a specimen between 30 and 100 hours was nearly constant, and the rate of flow from 40 to 50 hours was used as a basis for computing the efficiency of a coating according to the following equation:

$$\text{Percentage Efficiency} = \frac{\text{Loss in flow due to treatment}}{\text{Untreated flow}} \times 100.$$

This assumes, of course, that there was no change in the permeability of the specimen itself between the original test and the test after treatment.

DISCUSSION OF RESULTS

Surface treatments may be compared on the basis of percentage efficiency, as defined above, or on the magnitude of the treated leakage. On the former basis, the imperviousness of the specimen itself has to be taken into consideration, for when this is comparable to that of the coating itself, a poor efficiency results. Better indications of coating efficiency are secured when the specimens are relatively leaky. On the basis of magnitude of treated leakage, the latter may be arbitrarily rated by reference to a selected leakage of .002 gal. per sq. ft. per hour, which is about the lowest leakage at which water appeared on the outer surfaces of the specimens under test.

The average results obtained with the various compounds in this investigation are shown in Fig. 5, which gives both efficiency and average leakage.

The effect of weathering of the treatments over a 2-year period is shown in Fig. 6. All showed a drop in efficiency due to exposure except the sodium sulphate-barium chloride treatment.

CONCLUSIONS

The results of these tests have been analyzed* in the previous section and may be summarized as follows:

*The detailed analyses are not here published but appear as a part of the complete paper as noted on page 8.

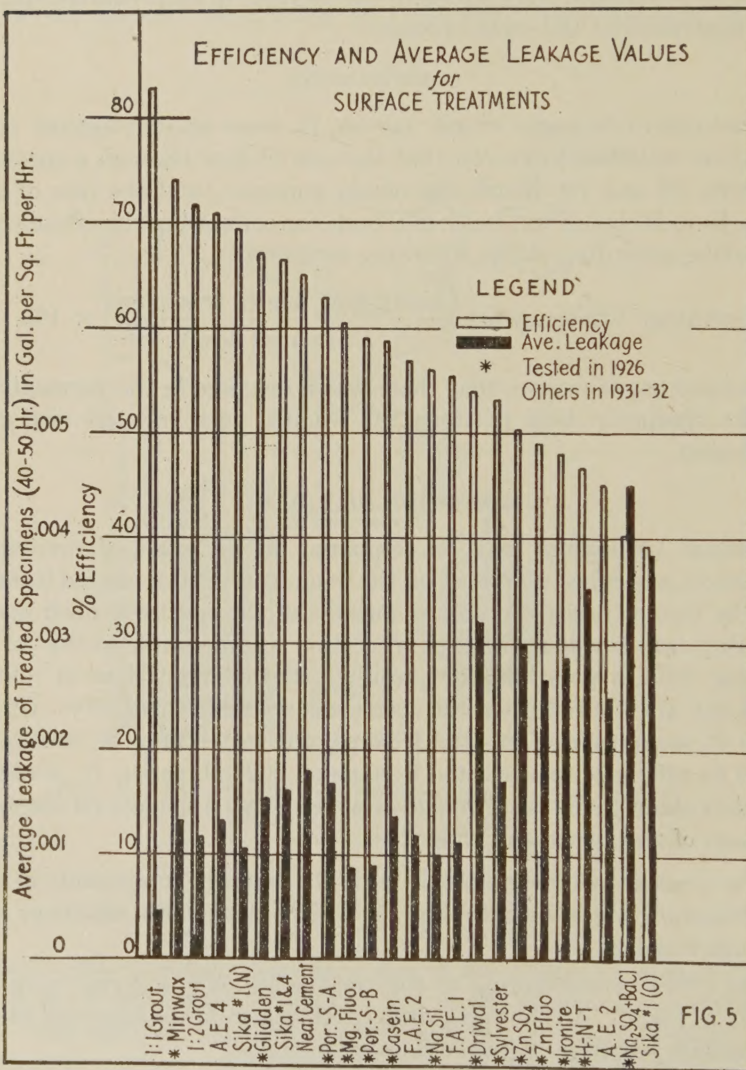

FIG. 5—EFFICIENCY AND AVERAGE LEAKAGE VALUES FOR SURFACE TREATMENTS

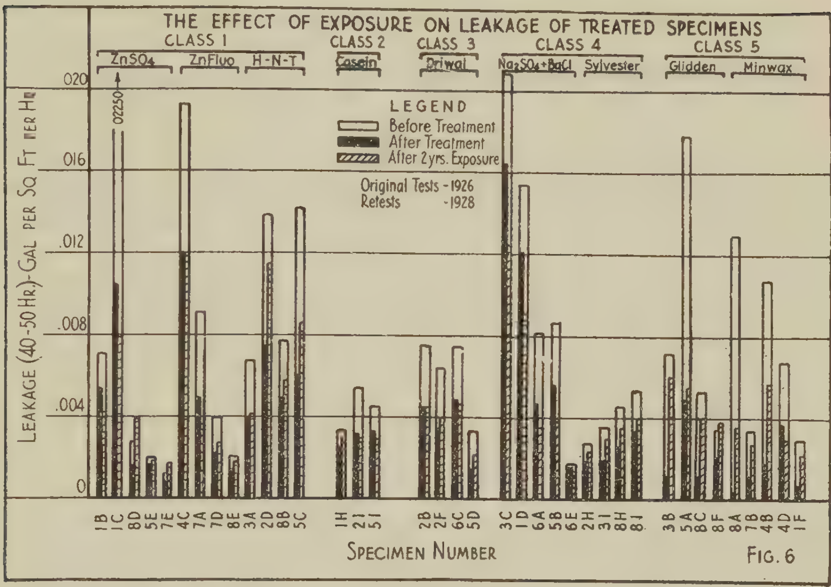

FIG. 6—EFFECT OF EXPOSURE ON LEAKAGE OF HEATED SPECIMENS

1. The use of surface treatments to obtain lower leakages through concrete was more or less beneficial in all cases. The value of the various treatments, however, varies widely, with some treatments giving an efficiency of around 40 per cent and others giving efficiencies very close to 90 per cent. The efficiency of the treatment depends upon the compound used and also on the method of application. That is, a compound properly applied giving good results might give very poor results if improperly applied. Also the efficiency is dependent more on the individual treatment used than on any class of treatment.

2. Some of the more effective types of treatments according to these tests are 1:1 grout properly cured, Minwax, 1:2 grout properly cured, neat cement properly cured, plain asphalt emulsion, Glidden, Sika grouts, Por-seal "A," Casein, and sodium silicate (see Fig. 2).

3. The rate of leakage through the specimen for the 20 to 50 hour period is practically the same as the rate for the 40 to 50 hour period.

4. If cement grouts are to be used they should be moist cured for at least a week to obtain the best results. Tests showed that the efficiency of a 1:1 grout was reduced from 88 per cent to 45 per cent by decreasing the moist curing period from seven to zero days.

5. The 1:1 grout cured for seven days gave an efficiency of 88 per cent which was the best of any of the grouts tested. With the leaner 1:2 grout the efficiency was 72 per cent and with neat cement it was only 65 per cent.

6. Exposure to the elements decreased the efficiency of all treatments tested with the exception of the sodium sulphate plus barium chloride treatment. The decrease in efficiency, caused by an exposure of two years, varied from 5 to about 80 per cent based on original efficiency. The treatment which was an exception to the general trend increased its efficiency about 70 per cent, also based on original efficiency.

The foregoing contribution is the author's summary of his complete report which will be made available by the Institute to readers of this JOURNAL. It will be sent to members postpaid on receipt of 50 cents and to non-members for $1.00 per copy. The summary here presented contains "Introduction" and "Conclusions" in full. The original detailed descriptions under the sub-heads, "Materials and methods of application," "Testing apparatus," "Testing procedure," "Computations," and "Discussion of results," are here briefly summarized. The complete paper also carries three additional figures—one from a photograph of the testing apparatus (Fig. 1) and two "leakage time curves," (Fig. 3 and 4).

For such discussion of this paper as may develop readers are referred to the JOURNAL for March-April, 1934. Discussion to be acceptable should be based upon the full report as mentioned above and be available to the Secretary of the Institute by February 1, 1934.—EDITOR

Comparison of Selected Portland Cements in Mass Concrete Tests

Following are five of a group of eight papers, presented through the courtesy of the Bureau of Reclamation at the Institute's 29th Annual Convention, Chicago, February 21-23, 1933. Two of the papers were published in this Journal *for March-April, 1933 ("Mass Concrete Research for Hoover Dam," by Byram W. Steele, Proceedings, Vol. 29, p. 305; "An 8-Hour Accelerated Strength Test for Field Concrete Control," by O. G. Patch, Proceedings, Vol. 29, p. 318) and one in the* Journal *for June 1933 ("Cement Investigations for the Hoover Dam," by Raymond E. Davis, R. W. Carlson, G. E. Troxell and J. W. Kelly, Proceedings, Vol. 29, p. 413). All these papers are summaries only of extensive investigations which were not complete when the papers were prepared. It is planned that a complete report of all concrete research for Boulder (formerly Hoover) Dam will be published by the Bureau of Reclamation at a later date, at which time appropriate announcement will be made in this* Journal. *Readers are referred to the* Journal *for March-April 1934 for discussion as it may develop of this group of papers. Such discussion should be available to the Secretary of the Institute on or before February 1, 1934.*—Editor

BY ROBERT F. BLANKS*

INTRODUCTION

The control of volume change occasioned by temperature variations forms one of the most important problems in the design and placement of mass concrete for large hydraulic structures. These changes originate from two sources: (1) initial temperature resulting from atmospheric conditions and (2) heat generated by the hydration reaction of cement. Initiation of the series of tests for the comparison of cements received first impetus as a result of the need for additional information in order to arrive at a satisfactory solution of the volume change problem, with due consideration being given to such essential requirements as durability, strength development and elastic and plastic properties.

Obviously, variations in temperature cannot be eliminated, and it therefore becomes necessary to provide contraction joints or induced cracks. Shrinkage of the concrete under controlled conditions can

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then be obtained by the artificial extraction of excess heat, and grouting the resultant joint openings will tend to weld the structure into one continuous monolithic unit which is so essential to the transfer of stress as assumed in the design. Under this scheme, some excess heat is necessary to insure sufficient contraction for the formation of joint openings which can be successfully grouted after cooling has been effected to slightly less than mean annual temperature.

With a given set of atmospheric conditions and the adopted plan of construction as outlined above, the most logical point of first attack in the attainment of the desired volume change and mass concrete properties is the study and control of the chemical composition of the cement. Interpretation of earlier researches in cement and concrete indicated that the possibilities in this direction were very promising but yielded little direct and quantitative data for practical application. A comprehensive series of concrete tests was accordingly outlined to show the relation between chemical constitution of the cement and physical properties of the concrete and to define the practical limits of control through the cementing medium.

DETAILS OF THE TEST PROGRAM

The desire to duplicate field conditions in the laboratory as far as possible advised experiments to determine heat generation and other characteristics, not upon the cements alone, but upon representative specimens of mass concrete, cured and tested in close approximation to job stipulations. The apparatus and test methods developed for this series of concrete tests have been governed chiefly by the above considerations.

Based on the data obtained from the cement investigations conducted in the Materials Testing Laboratory at the University of California,[1] seven cements covering the desired range of cheminal composition and fineness were chosen for final study in concrete tests in Denver. These cements were specially burned and ground to approximate a predetermined composition and degree of fineness in laboratory equipment and in sufficient quantities to supply the requirements of the test program. In addition to these special cements, a number of commercial products have been or will be subjected to a similar cycle of tests.

The aggregates used in the fabrication of the concrete specimens for the comparative study of all cements as outlined below were obtained from the Arizona gravel deposit at Boulder Damsite.

[1]See paper by R. E. Davis, R. W. Carlson, G. E. Troxell and J. W. Kelly, "Cement Investigations for Hoover Dam," Journal, Amer. Concrete Inst., June 1933; *Proceedings*, Vol. 29, p. 413.

Temperature Rise and Heat Generation. Appropriate size cylinders of full mass mix concrete containing one barrel of cement per cubic yard and including cobbles are cast in place in accurately controlled adiabatic calorimeter rooms and immediately sealed by soldering a cover on the light sheet metal mold. (Adiabatic as here used indicates no loss of heat from the specimen and no gain of heat except that resulting from the chemical reaction of the cement.) Resistance thermometers are inserted in wells formed by light copper tubing sealed in the metal covers of the mold. One of these thermometers together with a matched bulb suspended in the adjacent air of the calorimeter room is connected to specially designed, sensitive, automatic temperature control instruments.[2] The hydration of the cement in the concrete generates heat which in turn causes a temperature rise in the mass. The control instruments operate to maintain the temperature of the air in the calorimeter room the same as that of the specimen so that adiabatic conditions obtain. The special design of the rooms insures uniformity of temperature throughout and the controls provide equality of temperature between the air and the specimen within 1/10° F. The time-temperature curve obtained is thus a measure of the heat generated by the cement and the conversion is easily made from a knowledge of the physical and thermal properties of the materials involved.[3]

The adiabatic curing of the sealed specimens closely approximate the actual field conditions since the great mass of the dam and the rapid rate of its construction tend to prevent appreciable loss of the heat from the interior of the concrete for some considerable time after placing and loss of moisture is restricted to a relatively thin outer shell. Also, the precisely controlled temperature cycle affords an accurate basis for conducting the tests and comparing the results. The temperature rise cycles are started from initial temperatures of 40°, 70° and 100° F. for all cements in order to cover the range of placing conditions expected on the job.

Compressive Strength and Elastic Properties. Specimens for obtaining the compressive strength and elastic properties are cast in sealed metal molds simultaneously with the temperature rise cylinder and are stored in the same calorimeter room. These include 2 by 4-in. plastic mortar cylinders using Arizona sand graded from 0 to No. 4; 3 by 6-in. concrete proportioned 1:5.20 by weight with ¾-in. maximum size aggregate and having the same surface area, w/c ratio and consistency as the full mass mix; 6 by 12-in. concrete fabricated from the mass mix wet-screened to a maximum size of 1½ in.; and 18 by 36-in. full mass concrete the same as used in the control specimen. Companion specimens not sealed are cast for curing in fog-rooms under standard conditions. All cylinders are tested for compressive strength at various ages and a part are also tested for Young's modulus and Poisson's ratio. A comparison of these properties for standard and mass curing is thus obtained for the various types of test pieces. The duration of the temperature rise test is usually 28 days and the strength specimens are stored (sealed) thereafter in a curing-room maintained at 70° F.[4]

Volume Change and Plastic Flow. Other specimens are placed in the calorimeter rooms for the determination of volume change and plastic properties. The former consist of 4 by 4 by 40-in. bars fabricated from the mass concrete after wet-screening to 1½ in. maximum size aggregate. Invar steel gage blocks in the ends of the bars

[2]A more complete description of the apparatus used in the temperature rise tests is contained in the paper, "Development of Large Calorimeter Rooms and Automatic Temperature Controls for Adiabatic Curing of Mass Concrete," by H. S. Meissner, in this JOURNAL.

[3]Details of the tests for thermal properties of mass concrete are given in the paper, "Thermal Properties of Mass Concrete," by C. S. Rippon and L. J. Snyder, in this JOURNAL.

[4]These tests will eventually be correlated with those described in the paper "Mass Concrete as Affected by Size of Aggregate and Related Factors," by Arthur Ruettgers, in this JOURNAL.

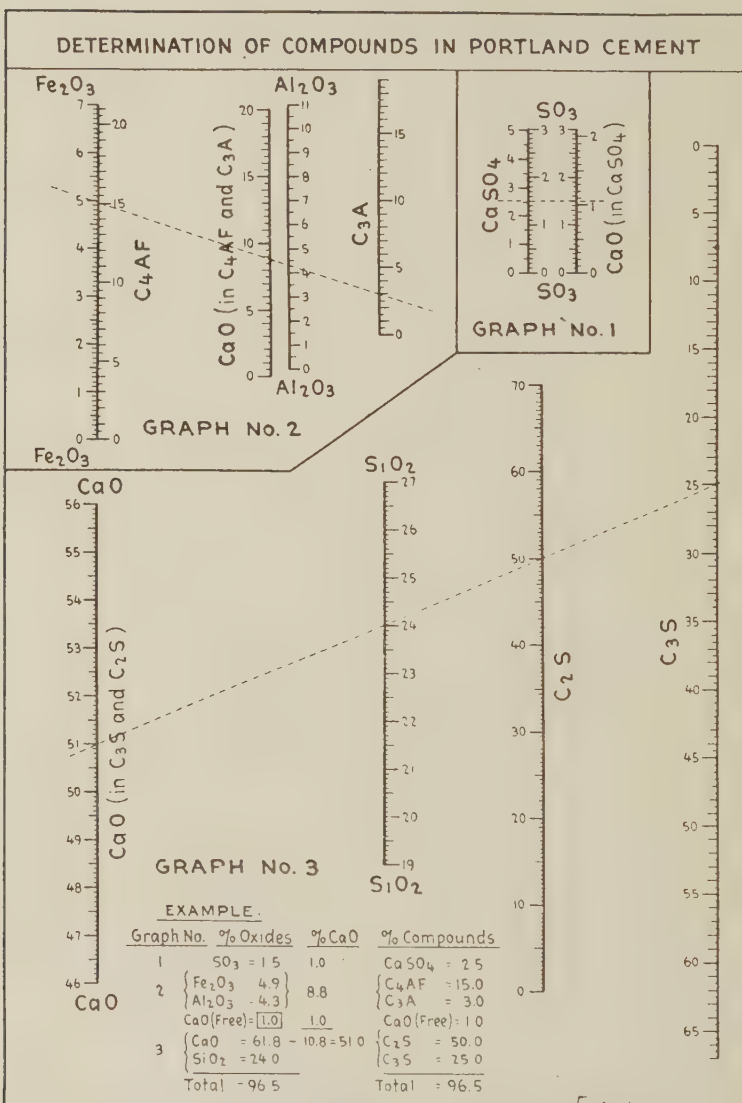
DETERMINATION OF COMPOUNDS IN PORTLAND CEMENT
GRAPH No. 1
SO3
CaSO4
CaO (in CaSO4)
GRAPH No. 2
Fe2O3
C4AF
CaO (in C4AF and C3A)
Al2O3
C3A
GRAPH No. 3
CaO
CaO (in C3S and C2S)
SiO2
C2S
C3S
EXAMPLE.
Graph No. % Oxides % CaO % Compounds
1 SO3 = 1.5 1.0 CaSO4 = 2.5
2 Fe2O3 4.9, Al2O3 4.3 8.8 C4AF = 15.0, C3A = 3.0
CaO(Free) = 1.0 1.0 CaO(Free) = 1.0
3 CaO = 61.8 − 10.8 = 51.0, SiO2 = 24.0 C2S = 50.0, C3S = 25.0
Total = 96.5 Total = 96.5
FIG. 1

FIG. 1

form reference points for observing length changes with a specially constructed frame caliper. The bars are sealed in thin copper jackets and at the end of the adiabatic curing period are removed to 70° F. curing-rooms. Part of the specimens are left sealed while the remainder are stripped of their containers for storage both under water and in air at 50 per cent relative humidity.

The flow specimens are also cast in thin copper containers and consist of 6 by 48-in. cylinders loaded by means of car springs for unit stresses of 200, 400 and 600 p. s. i. An ordinary hydraulic automobile jack equipped with a special dial gage and accurately calibrated in a testing machine supplies the means of loading to the proper stress and for subsequent checking and adjustment. Invar steel inserts over the length of the specimens and along three elements spaced 120 deg. apart are provided for strain gage observations. Various ages of loading are included in the test program. Each specimen will eventually be sawed into four test pieces for final testing for elastic properties and compressive strength.

Permeability. Sealed 12 by 12-in. specimens composed of the full mass mix concrete with the maximum size of aggregate limited to about 4½ in. provide data relative to the effect of type of cement on the permeability under conditions closely approximating those expected to obtain in the dam. These tests will also yield valuable information as to the effect of curing conditions and placing temperature on the relative water-tightness of mass concrete.

Physical and Chemical Properties. In addition to the investigations as outlined above, each cement is subjected to exhaustive tests for various chemical and physical characteristics.

First, the w/c ratio required to produce a 3-in. slump in the mass concrete is determined so that the consistency of the concrete for all cements will be maintained constant. The w/c ratio thus obtained is used in all concrete and mortar tests. Standard determinations are made for normal consistency and time of set. Fineness is obtained as the per cent passing the No. 200 and 325-mesh sieves and also in terms of surface area in sq. cm. per gm. by means of a turbidimeter apparatus. The standard test for soundness is made and amplified to include 1 by 1 by 6-in. neat cement bars for observing length changes with a special dial caliper. Specific gravity is obtained by means of the le Chatelier flask. Finally, the oxide composition is obtained from which the potential compound composition is computed using the method as outlined by Bogue. Correlation of the compound composition with various physical and concreting properties of the cements form a step in the analysis of the test data. A graphical solution of Bogue's method of conversion has been developed for rapidly converting chemical analysis to compound composition. The usefulness of this chart is greatly increased by the fact that it can also be used for converting the compounds to oxides as is required in the design of cement compositions (See Fig. 1).

SOME TEST RESULTS

The burning and grinding of the laboratory cements have but recently been completed and consequently the tests on these materials are just being started. However, numerous data on various commercial cements have been obtained, and the discussion of the test data thus far secured will be limited to these cements.

The curves of Fig. 2 and 3 show typical temperature rise and heat generation characteristics for a number of commercial cements covering a wide range in chemical compositions. In general, the higher heat

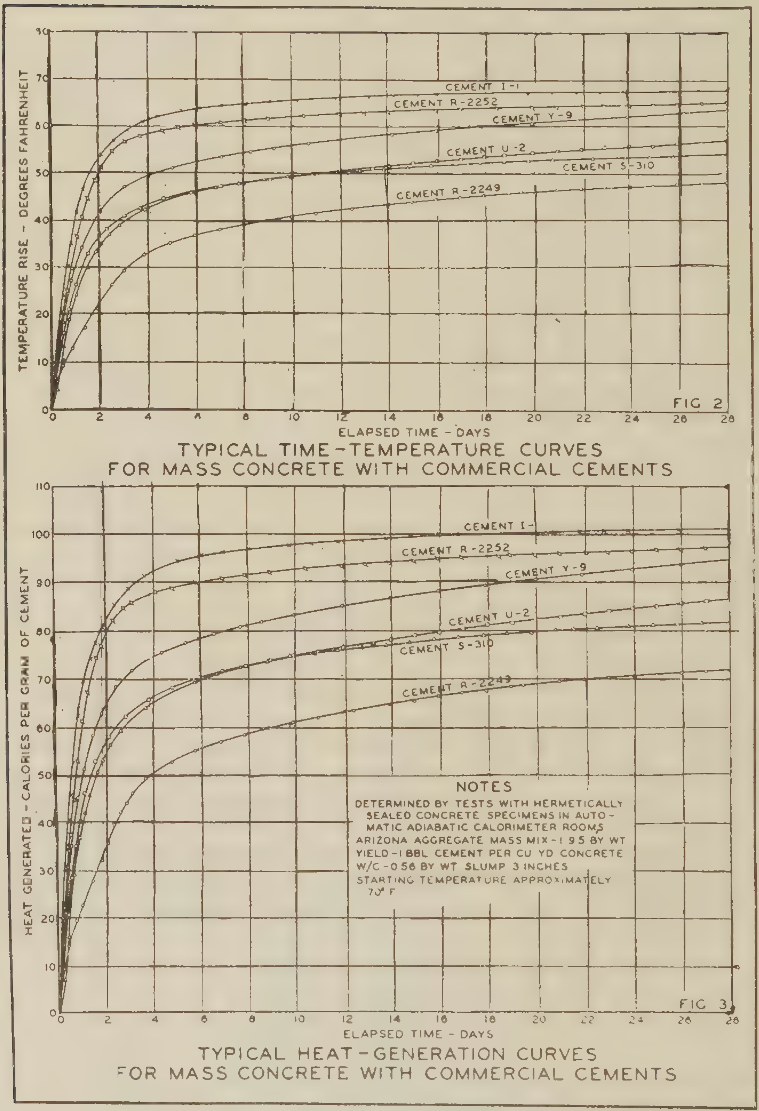
TEMPERATURE RISE - DEGREES FAHRENHEIT
CEMENT I-1
CEMENT R-2252
CEMENT Y-9
CEMENT U-2
CEMENT S-310
CEMENT R-2249
FIG 2
ELAPSED TIME - DAYS
TYPICAL TIME-TEMPERATURE CURVES
FOR MASS CONCRETE WITH COMMERCIAL CEMENTS
HEAT GENERATED - CALORIES PER GRAM OF CEMENT
CEMENT I-1
CEMENT R-2252
CEMENT Y-9
CEMENT U-2
CEMENT S-310
CEMENT R-2249
NOTES
DETERMINED BY TESTS WITH HERMETICALLY SEALED CONCRETE SPECIMENS IN AUTOMATIC ADIABATIC CALORIMETER ROOMS
ARIZONA AGGREGATE MASS MIX-1 9 5 BY WT
YIELD-1 BBL CEMENT PER CU YD CONCRETE
W/C-0 56 BY WT SLUMP 3 INCHES
STARTING TEMPERATURE APPROXIMATELY 70° F
FIG 3
ELAPSED TIME - DAYS
TYPICAL HEAT-GENERATION CURVES
FOR MASS CONCRETE WITH COMMERCIAL CEMENTS

FIG. 2 AND 3

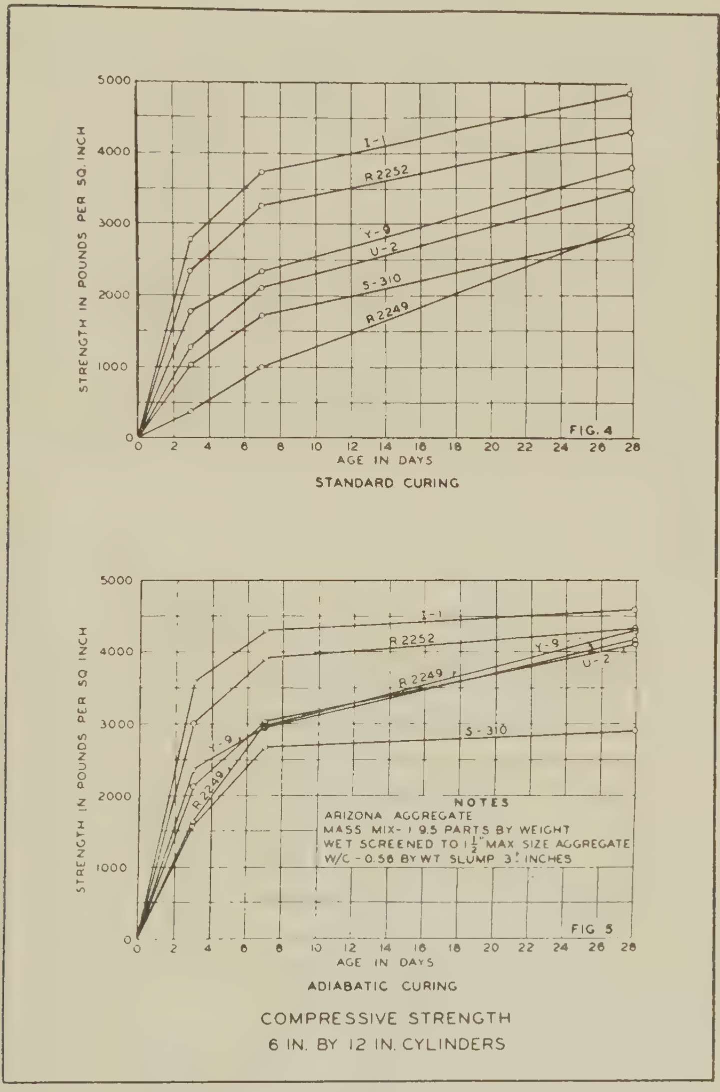
STRENGTH IN POUNDS PER SQ. INCH
AGE IN DAYS
STANDARD CURING
FIG. 4
I-1
R 2252
Y-9
U-2
S-310
R 2249
ADIABATIC CURING
FIG 5
NOTES
ARIZONA AGGREGATE
MASS MIX- 1 9.5 PARTS BY WEIGHT
WET SCREENED TO 1½" MAX SIZE AGGREGATE
W/C - 0.56 BY WT SLUMP 3½ INCHES
COMPRESSIVE STRENGTH
6 IN. BY 12 IN. CYLINDERS

Fig. 4 and 5

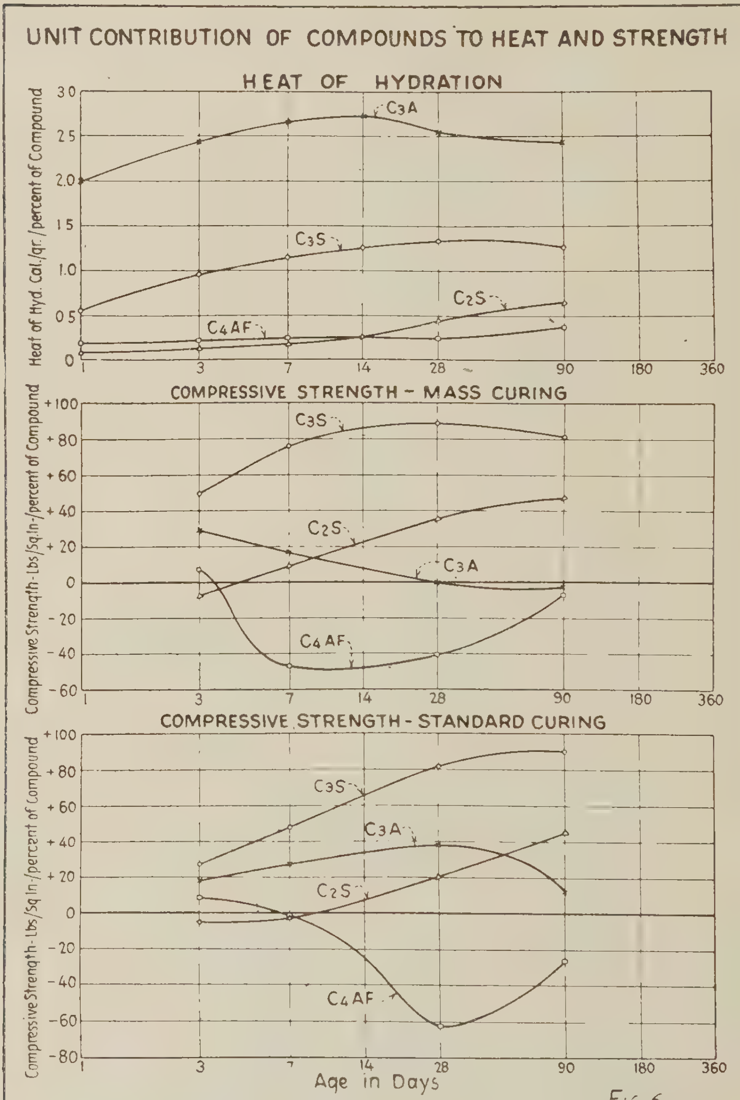
UNIT CONTRIBUTION OF COMPOUNDS TO HEAT AND STRENGTH
HEAT OF HYDRATION
Heat of Hyd. Cal./gr./percent of Compound
C3A
C3S
C2S
C4AF
COMPRESSIVE STRENGTH - MASS CURING
Compressive Strength - Lbs/Sq.In./percent of Compound
COMPRESSIVE STRENGTH - STANDARD CURING
Age in Days
FIG. 6

FIG. 6

TABLE 1—OXIDE AND POTENTIAL COMPOUND COMPOSITION OF COMMERCIAL CEMENTS REPRESENTED IN FIG. 2 TO 5

Cement No.	Chemical Analysis								
	SiO_2	Al_2O_3	Fe_2O_3	CaO	MgO	SO_3	Loss	Total	Insoluble
I-1	20.52	7.32	3.40	65.70	0.71	1.65	0.68	99.98	——
R-2252	21.26	4.62	3.28	64.54	3.60	1.51	1.17	99.98	——
Y-9	23.16	4.54	1.94	65.56	1.56	1.37	1.50	99.63	0.17
U-2	23.52	6.16	2.04	62.10	3.45	1.70	0.89	99.86	——
S-310	32.80	7.30	2.69	51.90	1.65	1.58	1.90	99.82	18.10
R-2249	22.56	4.86	5.94	59.40	4.07	1.42	1.02	99.27	——

Cement No.	Compound Composition								
	C_3S	C_2S	C_3A	C_4AF	MgO	$CaSO_4$	Free Lime	Loss	Total
I-1	49.1	21.9	13.6	10.3	0.7	2.8	0.9	0.7	100.00
R-2252	57.8	17.4	6.7	10.0	2.6	3.6	1.2	0.8	100.00
Y-9	52.9	26.5	8.8	5.9	1.6	2.3	——	1.5	99.50
U-2	23.2	50.1	12.9	6.2	3.5	2.9	0.3	0.9	100.00
S-310	Not a true Portland cement								
R-2249	24.6	46.2	2.8	18.1	2.4	4.1	1.0	0.1	99.30

cements are those containing high percentages of tricalcium silicate and tricalcium aluminate and a correspondlingly low percentage of dicalcium silicate while the low heat products are those for which the reverse is true. Apparently the compound composition is the major factor governing the heat generating characteristics of a cement although fineness of grinding and heat treatment during manufacture have appreciable effects. The upper curves in Fig. 2 and 3 represent one of the highest heat materials and is characteristic of the modern high early strength cement. The lower curve is near the lower limit of a low-heat cement that will also possess the necessary degree of strength development and which will be practical of commercial production.

Fig. 4 and 5 present the strengths obtained with 6 by 12-in. concrete cylinders for both standard and mass curing. It will be seen that high or low heat of hydration is in general indicative of correspondingly high and low compressive strengths. It will also be seen that at 28 days the strengths developed by the mass-cured cylinders are considerably higher than for standard curing. However, the indications are that at 90 days the difference is materially reduced and at even later ages the reverse is more generally the rule. In this respect the results of the concrete tests are in close accord with the cement tests. Another important fact indicated by these tests is the consistent gain in strength over a long period of the low heat cements as compared with the high early strength development and subsequent slow gain of the higher heat cements.

The effect of chemical composition upon the heat of hydration and compressive strength is demonstrated by the diagrams shown in Fig.

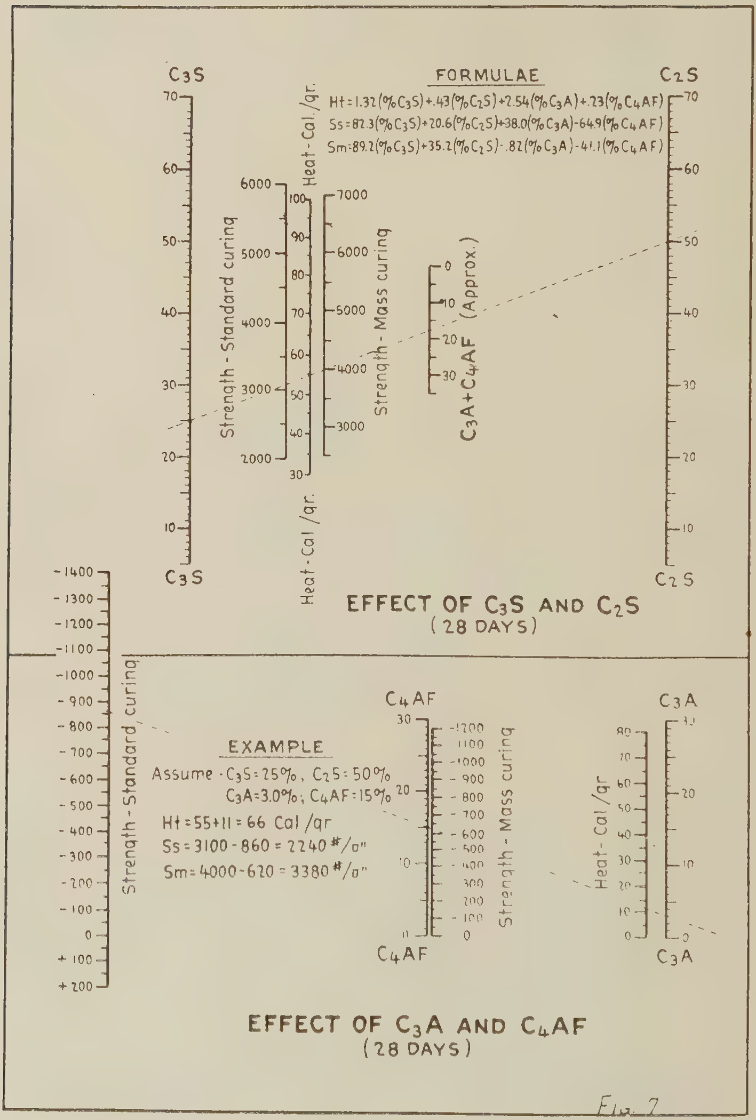
FORMULAE
Ht=1.32(%C3S)+.43(%C2S)+2.54(%C3A)+.23(%C4AF)
Ss=82.3(%C3S)+20.6(%C2S)+38.0(%C3A)-64.9(%C4AF)
Sm=89.2(%C3S)+35.2(%C2S)-.82(%C3A)-41.1(%C4AF)
C3S
C2S
Strength - Standard curing
Heat - Cal./gr.
Strength - Mass curing
C3A+C4AF (Approx.)
EFFECT OF C3S AND C2S
(28 DAYS)
EXAMPLE
Assume - C3S=25%, C2S=50%
C3A=3.0%; C4AF=15%
Ht=55+11=66 Cal/gr
Ss=3100-860=2240 #/□"
Sm=4000-620=3380 #/□"
C4AF
C3A
EFFECT OF C3A AND C4AF
(28 DAYS)
Fig. 7

FIG. 7

TABLE 2—HEAT GENERATION IN CALORIES PER GRAM FOR SOME COMMERCIAL AND LABORATORY CEMENTS TESTED IN MASS CONCRETE SPECIMENS CURED UNDER ADIABATIC CONDITIONS

Cement No.	1 Day		3 Days		7 Days		14 Days		28 Days	
	Experimental	Computed	Experimental	Computed	Experimental	Computed	Experimental	Computed	Experimental	Computed
I-1	62.4	59.5	88.4	85.2	96.2	100.2	99.1	107.8	101.0	110.8
L-24	48.9	50.4	75.3	74.9	86.9	89.0	89.8	95.7	92.5	101.0
U-2	40.1	45.0	62.2	61.3	71.6	72.1	78.0	79.1	86.2	85.9
L-23	34.6	40.7	49.9	53.2	58.1	62.1	65.9	69.2	——	77.2
R-2249	21.1	35.3	44.2	47.8	57.0	56.3	64.8	62.7	71.8	69.9

6 which represents in graphical form the unit contribution of the various compounds to the heat of hydration and strength development. These curves were prepared from an analysis of the data obtained in the cement investigations conducted at the University of California[1] and apply for laboratory cements having specific surfaces of approximately 1200 sq. cm. per gm. when used in plastic mortars having water-cement ratios of 0.56 to 0.60 by weight. Tricalcium aluminate (C_3A)is by far the largest contributor toward heat liberation yet offers but little aid in strength development. Tricalcium silicate (C_3S) supplies most of the early strength-giving quality while dicalcium silicate (C_2S) contributes materially toward this characteristic at the later ages. These diagrams aid in forming some conception of the relation between chemical composition and other properties as illustrated in Fig. 2 to 5. (See Table 1 for the oxide and compound compositions of the cements represented in Fig. 2, 3, 4, and 5.)

One of the more important objects of the concrete tests on various cements is the determination of the relation between the properties of cements as indicated by the tests with small specimens in the cement investigations and the properties exhibited in concrete as determined by the concrete investigations. For this purpose, the data from the Berkeley tests have been analyzed for various ages as illustrated in Fig. 7. A set of similar nomographs has been developed for all ages from 3 to 90 days and represents the first attempt to obtain a practical means of predicting the heat-generating and strength-developing characteristics of a cement from its compound composition. It should again be borne in mind that Fig. 7 applies for mortar tests on laboratory cements having a specific surface of about 1200 sq. cm. per gm. It has been gratifying to observe that the concrete tests thus far completed have shown fairly close agreement as to heat of hydration with the expected values as computed from nomographs exemplified by Fig. 7, after giving due consideration to such variable factors as heat treatment of the cement clinker, special compositions, method of test, fineness, etc. (See Table 2). The agreement between actual strengths

and the predicted strengths as computed from Fig. 7 has not been as satisfactory as for heat of hydration. It is believed, however that additional data will make possible a revision of the factors of computation so that a much better check will eventually be obtained for both strength and heat development.

The accumulation of data and analysis of the results from the other tests included in this investigation are as yet rather meager and do not warrant presentation at this time. Preliminary results are promising and will contribute materially to the general understanding and application of the properties of portland cement when used in mass concrete.

See Editor's note top of page 9 in reference to discussion of this and other papers of this group.

Development of Large Calorimeter Rooms and Automatic Temperature Controls for Adiabatic Curing of Mass Concrete*

BY HARMON S. MEISSNER†

INTRODUCTION

The adiabatic curing of concrete test specimens to determine heat generation, volume change and other characteristics under conditions simulating those within a mass presents a problem that apparently has received attention only in the last year or two. Its solution in connection with concrete tests for the final comparison of cements for Boulder Dam introduces a design that is not only unique, but offers new thought and methods in the field of automatic temperature and heat control. For this purpose a room is required in which the temperature can be maintained in close agreement with that of a contained concrete control specimen, to the end that no heat will be lost or gained by the concrete.

After a thorough investigation of the possible means of maintaining adiabatic conditions (No loss of heat from the specimen and no gain of heat except that resulting from the chemical reaction of the cement) for curing mass concrete specimens, the type of room and control apparatus herein described was selected and developed. To accommodate a considerable number of specimens of various sizes and shapes, a room having a floor area of approximately 80 sq. ft. was required, making it difficult to secure uniform temperature distribution except by the novel circulation system adopted. The undesirable heating effects of fans or motors placed directly in the room was overcome by housing all air conditioning apparatus in a separate duct and excluding all operating machinery, with the exception of the blower, from either chamber. Electrical energy was chosen for the heat supply because of its instantaneous response to modulation and, consequently, its adaptability to automatic control.

*Presented at the 29th Annual Convention, Chicago, Feb. 21-23, 1933. See Editor's note, top p. 9.
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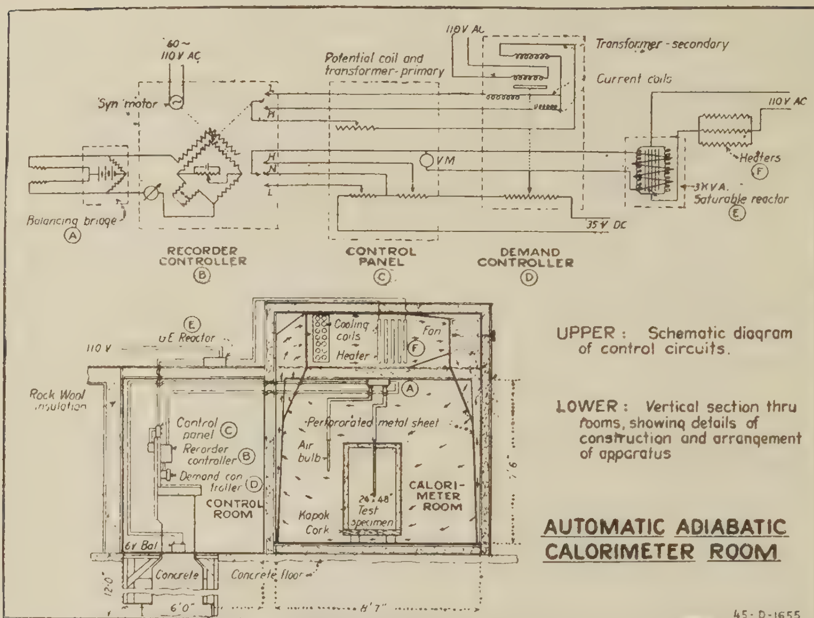

60 ~
110 V AC
Syn motor
Balancing bridge
(A)
RECORDER
CONTROLLER
(B)
Potential coil and
transformer-primary
CONTROL
PANEL
(C)
110 V AC
Transformer - secondary
Current coils
VM
35 V DC
DEMAND
CONTROLLER
(D)
110 V AC
Heaters
(F)
3 KVA.
Saturable reactor
(E)
(E)
GE Reactor
110 V
Rock Wool
insulation
Cooling
coils
Heater
Fan
(F)
(A)
Perforated metal sheet
Air
bulb
Control
panel (C)
Recorder
controller (B)
Demand con
troller (D)
CONTROL
ROOM
CALORI-
METER
ROOM
24×48"
Test
specimen
Kapok
Cork
6 V Bal
Concrete
Concrete floor
12'0"
6'0"
8'7"
UPPER: Schematic diagram
of control circuits.
LOWER: Vertical section thru
rooms, showing details of
construction and arrangement
of apparatus
AUTOMATIC ADIABATIC
CALORIMETER ROOM
45-D-1655

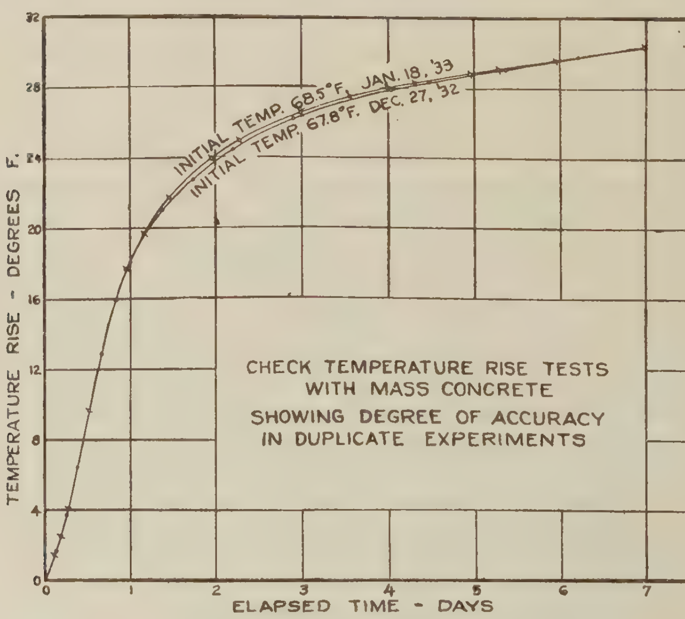

INITIAL TEMP. 68.5°F. JAN. 18, '33
INITIAL TEMP. 67.8°F. DEC. 27, '32
CHECK TEMPERATURE RISE TESTS
WITH MASS CONCRETE
SHOWING DEGREE OF ACCURACY
IN DUPLICATE EXPERIMENTS
TEMPERATURE RISE - DEGREES F.
ELAPSED TIME - DAYS
0
4
8
12
16
20
24
28
32
0
1
2
3
4
5
6
7

FIG. 1 AND 2

CALORIMETER ROOM DESIGN

Structural features. Fig. 1 shows a cross section through the control room and one of the adiabatic calorimeter rooms, together with the schematic arrangement of the automatic electrical control apparatus. All concrete specimens are hermetically sealed in galvanized sheet metal containers to prevent moisture loss and the room is lined with the same material with all seams soldered to insure air-tightness and to minimize radiation losses. Four to six inches of rock wool is used to insulate the walls, floor and ceiling which are so constructed that little framing material is continuous between the inside and outside surfaces.

The control room, housing the temperature control and recording apparatus, is also insulated to provide uniform temperature conditions for the instruments. A vibrationless table mount is provided for the sensitive control instruments, to absorb and reduce traffic tremors that would otherwise deflect the galvanometer needles.

Circulation system. The requirement that all concrete specimens in a room follow essentially the same temperature cycle as the master control specimen at all times necessitated virtually equal temperature distribution throughout the room and therefore a high degree of air circulation. Air in the room is changed completely four times a minute, being drawn out one side of the room into a duct overhead, over cooling coils and electrical heater elements, through a multiblade blower and again forced into the room on the opposite side. There are some 3000 $\frac{3}{16}$ in. round bell-mouthed ports of entry and exit over the entire area of each side of the room. The air diffuses into and flows across the room with such uniformity and rapidity that dead air spaces are eliminated and the air entering is at practically the same temperature as that leaving, even when there is demand for the addition or subtraction of considerable heat.

Air conditioning. Refrigeration is secured by circulating chilled brine through fin type coils in the conditioning duct, pumping it from a tank where it is held to 25° F. by immersed ammonia coils. Control is manual and is accomplished by valving the brine flow. Whenever it is necessary to extract heat from the room a slight excess of refrigeration is applied and heat then added by the automatic control to balance this excess. Automatic temperature control is thus obtained through the medium of electrical heaters which, by their instantaneous action, prevent appreciable lag or overshoot. Any necessity for controlling air humidity is obviated by hermetically sealing the test specimens.

TEMPERATURE DIFFERENCE CONTROL APPARATUS

To reproduce absolutely the time-temperature conditions existent in mass concrete perfectly protected against heat losses, a zero temperature difference must be maintained between the concrete specimen and the calorimeter throughout the test period. Obviously any control apparatus can only approximate the theoretically ideal condition, however, excluding the effect of minor heat losses through absorption by the specimen container, the scheme developed will hold the temperature difference within 0.1° F. or less and so operate that periods of temperature difference will tend to balance each other. In other words, for any periods in which heat flows from the concrete there will be other equal periods during which heat will be returned. A slight amount of insulating material is placed around the specimen to iron out the effect of transient inequalities. The heat required to raise the temperature of the metal container and one-half of the insulation around the specimen is provided by the addition to the concrete of excess cement above that required by the nominal mix. Thus, if the concrete normally contains one barrel of cement per cubic yard, the mix used in the control specimen is made richer by the addition of about 4.5 per cent more cement and the corresponding amount of water.

The apparatus for controlling temperature difference consists of four distinct parts: (1), a detector device for detecting temperature differences between the concrete specimen and the air and relaying such differences to (2), a recorder-controller which traces a continuous record of the temperature conditions and operates to overcome inequalities by actuating (3), a demand-controller that gages the supply of heat required by the room and tends to balance plus and minus temperature differences through the medium of (4), a saturable reactor which acts as a valve on the flow of electrical energy through the room heater coils.

Detector device. The temperature difference detecting device consists of two pairs of matched electrical resistance coils in the calorimeter room connected in series to form a wheatstone bridge. Two are mounted as resistance thermometers whose resistances vary with changing temperatures. The other two are of constant resistances and do not change with temperature fluctuations. The bridge of the detector is connected with a sensitive galvanometer in the recorder-controller.

Recorder-controller. The recorder-controller is a standard pyrometer instrument of the null-point balancing potentiometer type with certain revisions to adapt it to the desired purpose. It consists essentially of a wheatstone bridge circuit, a galvanometer which is common to the detector device and the necessary mechanical and electrical connections for performing three functions.

A continuous record of temperature differences existing between the concrete specimen and the calorimeter room as reported by the detector is traced on a roll-chart. A considerable quantity of heat in the form of electrical energy is added to or detracted from the room upon the first indication of temperature difference, immediately to check such difference before it can grow to any sizable proportions. The demand-control is actuated slowly to add or detract heat as required by the room to the end that a zero temperature difference is maintained between the concrete and the calorimeter.

Demand Control. This instrument is a specially designed and constructed unit composed of a reversible, variable-speed disc-type motor which actuates a sliding contact upon direct current potentiometer, thereby controlling the D. C. voltage applied to a reactor.

Saturable Reactor. A standard commercial product is utilized and consists merely of two induction coils wound on a common iron core. One coil carries direct current regulated by the demand control and governs the reactance or degree to which the core is saturated with magnetism. The other coil is connected in series with the heater units, to an alternating current power supply. The amount of power flowing through this coil and hence through the heaters, is dependent upon the degree of saturation as fixed by the D. C. winding.

Operation as a Unit. For the purpose of illustrating the operation of the control apparatus as a whole, it will be assumed that the temperature of the calorimeter room air becomes slightly less than that of the concrete specimen, thus creating a temperature difference. The two resistance thermometer coils are then of unequal resistance, unbalancing the detector device bridge and causing the galvanometer contained in the recorder-controller to deflect from the neutral or zero temperature difference position. Deflection of the galvanometer needle sets in motion the mechanical parts of the instrument, moving the recording pen upon the chart and actuating mercury switches immediately to add an increment of heat to the room as well as start the disc motor of the demand control to rotate. This then operates to move the sliding contact on the potentiometer, increasing the flow of direct-current through the reactor, thus decreasing its reactance and permitting more power to flow to the heaters. The demand controller augments the heat supply at a constant rate until the temperatures of the air and concrete again become equal, when its action is stopped by the recorder-controller.

By the automatic action of the demand-controller a balance of temperature difference is always maintained. Should the supply of heat to the room be practically the correct normal amount and an additional demand be suddenly created, as by reason of opening the door, the disc motor will turn on a supply above this normal amount. Then when the door is again closed, the presence of too much heat will cause a temperature difference of the reverse order until the abnormality is corrected. Thus the two dissimilar periods will offset each other and result in a net time-temperature difference of approximately zero.

PERFORMANCE

A remarkably close temperature control has been achieved. Thus, while the room was being controlled to constant temperature, by reason of having the specimen thermometer in an aged inert concrete cylinder, room fluctuations were observed independently with an extremely sensitive resistance thermometer placed within the room and attached as one of four equal arms in a Wheatstone bridge. A sensitive mirror type galvanometer was used on the bridge and the deflections observed. Variations were found to be generally not more than .05° F. and never to exceed 0.1° F. between extremes. The control instrument was found frequently to respond to a temperature difference of .028° F. As an additional preliminary test for performance and precision of the apparatus, control under the conditions above mentioned was continued for several days without any perceptible raising or lowering of the room or specimen temperature.

To determine the degree of accuracy that might be expected in the way of duplicating results, a temperature rise test was, after several weeks, repeated in another calorimeter room. The same cement, thoroughly blended, was used on both experiments. Remarkable agreement was achieved, in that the temperature rises differed but little between the first and fifth days and exactly coincided by the sixth day. The slightly higher values obtained in the later test may be accounted for by a higher concrete temperature at the start of the test, which has been proved to accelerate heat generation in the early periods.

See Editor's note top of page 9 in reference to discussion of this and other papers of this group.

Mass Concrete as Affected by Size of Aggregate and Related Factors*

BY ARTHUR RUETTGERS†

INTRODUCTION

The purpose of these tests is to investigate the effect of size of aggregate, mix proportions and size of test specimen on compressive strength, elastic properties and permeability of mass concrete, as an aid:

(1) In analyzing the relative merit and economy of mass concrete with and without cobbles‡ of various sizes,

(2) In establishing the strength relation, for field control purposes, between representative full-mix concrete containing cobbles and the same concrete after being wet-screened or hand-picked to remove aggregate which, under standard specifications, is too large for inclusion in small field test cylinders,

(3) In determining, for design purposes, the unit strengths of mass concrete containing cobbles of various sizes,

(4) In judging the sufficiency and economy of the Bureau of Reclamation's previous general practice, in massive dam construction, of employing a minimum portland cement content of one barrel per cubic yard of concrete, and cobbles as large as can be handled through the mixer,

(5) In meeting mass concrete impermeability requirements, particularly at the upstream face of dams exposed to extraordinary heads, and

(6) In solving other miscellaneous problems.

The details of the test outline were arranged with particular reference to application of the test data to the problems of Boulder Dam.

The major portion of the investigation was made possible through the recent installation in Denver of a 4,000,000-lb. capacity hydraulic testing machine equipped with variable discharge pump and specially designed for breaking concrete cylinders up to 36 in. in diameter and 72 in. high. The machine, a part of which is shown in Fig. 3, is one of the largest and most modern concrete testing machines.

PRELIMINARY INVESTIGATIONS

As the test program involves a wide range in cylinder and aggregate sizes, and testing procedure for which existing standard specifications

*Presented at the 29th Annual Convention, Chicago, Feb. 21-23, 1933. See Editor's note, top p. 9.
†Engineer, U. S. Bureau of Reclamation, Denver, Colo.
‡Considered as coarse aggregate too large to pass a screen having 3-in. round or 2¾-in. square openings.

are largely inapplicable, a number of preliminary investigations were found necessary to insure uniform conditions of mixing, fabricating, curing and testing.

Rather extensive investigations were made to develop a method of compacting the concrete uniformly by vibration, to devise a satisfactory system of grinding the top surfaces of cylinders in lieu of capping, and to arrive at the most appropriate method of loading test cylinders.

The vibration experiments dealing with plastic concrete of average 3-in. slump consistency led to the conclusion that use of a vibrating spud bar or pipe in a certain manner provided uniform concrete compaction in all sizes of test cylinders. The improvement in uniformity of aggregate distribution and appearance of the matrix by vibration as compared with handrodding was especially noticeable. Little difference in compressive strength was found for the two methods of compaction. In general the vibrated concrete was slightly weaker.

The tests on the effects of cylinder end condition indicated that grinding with a suitable stone-polishing machine was the most satisfactory method of providing plane ends where a wide range in cylinder sizes was involved. For 6- by 12-in. cylinders it was also found that grinding, capping with molten sulfur and fire-clay compound, and capping with portland or Lumnite neat cement all resulted in about equal compressive strengths.

After a thorough study of the loading problem it was concluded that a uniform rate of unit load application for all sizes of cylinders was the most logical and satisfactory method of securing uniform conditions of testing. Special load-rate control apparatus was accordingly developed and a rate of loading of 17 p. s. i. per second was adopted. A comparison of the standard method of loading (.05 in. per minute head travel at idling speed) with methods of uniform load application developed the following:

(1) The standard method of loading results in wide variations in rate and total time of testing for equal unit breaking strengths in cylinders of the same or different sizes.

(2) Within practical limits of rate of uniform load application, uniform loading results in somewhat higher indicated strength than standard loading.

(3) Increasing the rate of uniform load application over a wide range increased the indicated strength of 6 x 12-in. cylinders consistently, although the total increase was less than 10 per cent.

It was originally intended to seal all test specimens in light metal containers to insure uniform moisture conditions in large and small cylinders. However, as comparative tests showed substantially equal strengths for sealed and unsealed curing of specimens of the same size, and as the added expense and labor of sealing was found to be considerable, the sealing method was abandoned. All specimens are stored in fog-rooms maintained at 70° F. ± 2° F. by specially designed and constructed air-conditioning units with automatic controls.

The volume production of concrete required by the test schedule necessitated machine-mixing, which in turn demanded the development of standardized procedure in all phases of the batching and mixing operations. The extent to which this standardization was carried might appear absurd if outlined in detail. However, there is evidence of justification for the procedure established, as the mean variations in strength of large groups of specimens seldom exceed 5 per cent, and are usually between 3 and 4 per cent.

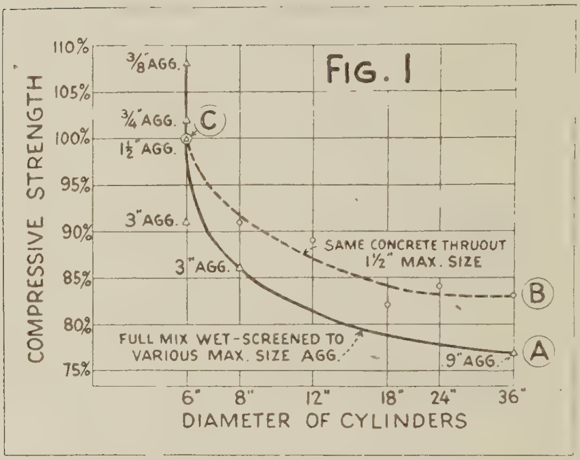
DETAILS OF MAIN SERIES OF TESTS

The investigation proper includes two comprehensive series of tests for compressive strength and elastic properties. The first series is designed to reveal the effects of varying maximum size of aggregate and mix proportions. Three basic mixes are provided, having 9-in. maximum size aggregate and cement contents of 1.2, 1.0, and 0.8 bbl. per cu. yd. of concrete. In addition to each basic mix there are four mixes of equivalent consistency and water-cement ratio and virtually the same aggregate surface area per unit of cement, and having 6-in., 3-in., 1½-in. and ¾-in. maximum size aggregate. The full mixes are tested in cylinders having a diameter of at least four times the maximum size of aggregate and the concretes are also wet-screened to 3 in. and 1½ in. for testing in 8-in. and 6-in. diameter cylinders respectively.

The second series is designed to show in more detail the relation between the full mass mix having one barrel of cement per cu. yd. of concrete and the partial wet-screened mixes. In addition, this series is aimed to indicate directly the effect due to size of test cylinder for identical concrete mixes. The full mix with 9-in. maximum size aggregate is tested in 36 by 72-in. cylinders and is also wet-screened to 6-in., 3-in., 1½-in., ¾-in., and ⅜-in. maximum for testing in all sizes of cylinders down to the minimum commensurate with each size of aggregate. Seven sizes of test cylinders ranging from 3 by 6 to 36 by 72-in. are included in this test series.

The program of tests for permeability embraces cylinders varying in size from 6 by 6 to 18 by 18-in. for testing under pressures up to about 450 p. s. i. The effects of mix proportions, maximum size of aggregate, type of cement and conditions of curing are being investigated. The specimens are tested in cast-steel containers and sealed with asphaltic material to prevent leakage around the specimen. Pressure is supplied from oxygen tanks and controlled by pressure-regulating valves. Water outflow as well as inflow is measured to obtain the complete time-flow history of each test.

Materials. One brand of a normal portland cement, carefully blended and stored in sealed 50-gal. steel drums, is used in all tests with the exception of the permeability



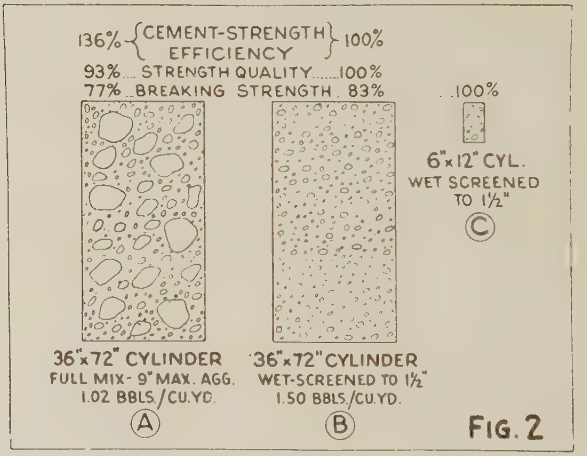


FIG 1 AND 2

tests. The aggregates used are from the so-called Arizona deposit, the source of supply for Boulder Dam. The pit is a typical water-borne deposit and contains material of excellent quality and almost ideal mass concrete gradation from 0 to 12 in. The sand is composed largely of quartz grains while the gravel is made up principally of limestone inter-mixed with granite, basalt and quartzite. The material is screened into twelve sizes from minus 100-mesh to 9-in. cobbles at the site and shipped to Denver in carload lots.

SOME TEST RESULTS AVAILABLE TO DATE

Fig. 4 shows a 36 by 72-in. cylinder after failure in compression. Attention is called to the double-cone break, a general characteristic noted for cylinders of all sizes. Typical cones taken from 36-in., 24-in., 12-in. and 8-in. diameter cylinders are shown in Fig. 5.

Another general aspect of the strength tests to date is that greater uniformity in strength may be expected from the larger cylinders. This is fortunate, since it tends to place the average strength value of a small number of large expensive cylinders on a par with a large number of the small relatively inexpensive specimens.

As the two main series of tests for strength and elastic properties are far from complete, it is too early to draw any final conclusions. Nevertheless, available test data show some trends which it is believed are of sufficient interest to warrant comment, even though they may not be wholly reliable.

Two curves are shown in Fig. 1 (see also Tables 1 and 2) to give some conception of the relative strengths of full-mix and wet-screened concretes, the concrete wet-screened to 1½-in. maximum size aggregate being considered as having 100 per cent relative strength. Beginning with point A at the extreme right of the full-line curve, which point represents the strength of a full-mix concrete with 9-in. cobbles in 36 by 72-in. cylinders, and proceeding toward the left it will be noted that the strength climbs as more and more of the mortar-coated coarse aggregate is removed and as the resultant concretes are tested in smaller and smaller cylinders. The large differences in strength cannot well be attributed wholely to differences in mix proportions resulting from wet-screening since, by examining the broken-line curve, which applies only to 1½-in. wet-screened concrete tested in cylinders of varying size, it is evident that the strength falls off materially as the size of test specimen increases.

The effects of cylinder size and wet-screening may be better visualized by considering only a few key-points on the curves, namely points A, B and C. The test specimens represented by these points are shown in Fig. 2. Specimen A is the large cylinder of full mass mix; specimen B is of the same size but made of the 1½-in. wet-screened concrete; while specimen C contains the same concrete as specimen B but cast in the regulation 6 by 12-in. cylinder. The relative breaking strengths of the three cylinders, as shown immediately above the cross-sections, are 100 per cent for specimen C, 83 per cent for specimen B, and 77 per cent for specimen A.

TABLE 1

Mass Mix Parts by Wt.	Maximum Size of Aggregate ((in.)	Size of Cyl. (in.)	Average 28-Day Strength: No. of Specimens	Average 28-Day Strength: Lb. per sq. in.	Average 28-Day Strength: % of 6″x12″ W.S. to 1½″	28-Day Elastic Properties: No. of Specimens	28-Day Elastic Properties: Young's Modulus	28-Day Elastic Properties: Poisson's Ratio
1:2.45:7.05 (1.02 bbls. per cu. yd.)	9″ W.S. to ⅜″	6x12	8	4340	108	4	4,500,000	.22
	9″ W.S. to ¾″	6x12	8	4100	102	4	5,000,000	.22
	9″ W.S. to 1½″	6x12	44	4020	100	8	5,300,000	.22
	9″ W.S. to 3″	6x12	8	3640	91	4	5,400,000	.22
	9″ W.S. to 3″	8x16	4	3450	86	4	5,400,000	.19
	9″ full mix	36x72	8	3100	77	7	5,200,000	.18
1:2.45:6.47 (1.07 bbls. per cu. yd.)	6″ W.S. to 1½″	6x12	18	4060	100	9	5,300,000	.20
	6″ W.S. to 3″	8x16	8	3560	88	6	5,600,000	.20
	6″ full mix	24x48	3	3080	76	2	5,200,000	.17

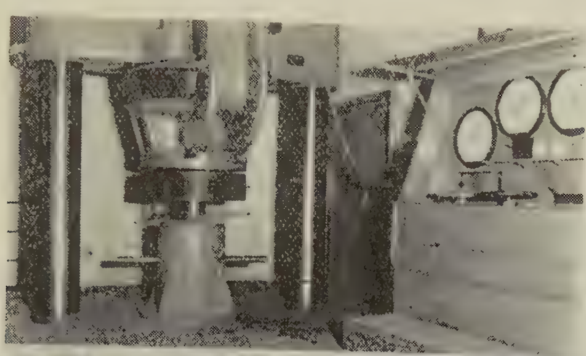
W/C = .544 by wt.
Slump = 3 inches ±.
W.S. signifies wet-screened.
Arizona aggregates with controlled grading.
Top surfaces of all specimens ground for test.
Specimens broken under uniform load application of 17 p. s. i. per sec.

TABLE 2

Mix Details	Size of Cyl. (in.)	Average 28-Day Strength: No. of Specimens	Average 28-Day Strength: Lb. per sq. in.	Average 28-Day Strength: % of 6″x12″	28-Day Elastic Properties: No. of Specimens	28-Day Elastic Properties: Young's Modulus	28-Day Elastic Properties: Poisson's Ratio
1:2.44:3.30 equivalent of wet-screening full mix of 1:9.5 and 9″ max. size to 1½″ max.	6x12	85	4050	100	4	5,200,000	0.21
	8x16	9	3680	91	3	5,100,000	0.19
	12x24	6	3590	89	2	4,500,000	0.18
	18x36	6	3320	82	—	—	—
	24x48	6	3400	84	2	4,900,000	0.20
	36x72	3	3380	83	2	5,000,000	0.20
1:2.45:7.05, 9″ max. full mass mix	36x72	8	3100	77	7	5,200,000	0.18

Assuming now that the concretes in specimens A and B may be legitimately compared for quality on the basis of their relative strengths in the large cylinders, then the strength quality of the cobble concrete is 77/83 or roughly 93 per cent of that of the wet-screened concrete. However, specimen A contains only 1.02 barrels of cement per cu. yd., as compared with 1.50 barrels for specimen B. Hence by dividing the relative strengths by the unit cement contents and comparing the results, it may be argued that the full mix concrete is about 36 per cent more efficient as a strength producer than the wet-screened concrete.

With respect to elastic properties, the trends of variation shown by the available test results are not so well defined or consistent. Two tendencies, however, may be worthy of mention: First, there is some indication (see Table 1) that wet-screening to a decreasing maximum size of aggregate and testing in the same size cylinder results in a fairly consistent decrease in Young's modulus; second, it appears (see Table



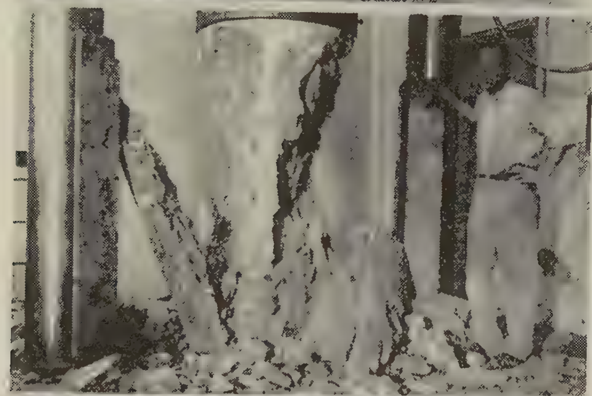


Fig. 3—4,000,000-lb. capacity compression testing machine

Fig. 4—Typical double-cone failure of 36 by 72-in. cylinder. mix 1-2.45-7.05 by wgt. with 9-in. aggregate. w/c = 0.81 by vol. 3-in. slump

Fig. 5—Typical cones from 36-in., 24-in., 12-in. and 8-in. cylinders

2) that for the same concrete, the size of cylinder has no pronounced effect on either Young's modulus or Poisson's ratio. This second tendency, combined with the fact that elasticity determinations to date have invariably shown straight-line stress-strain relations,* indicates that when identical concrete is tested under uniform loading in cylinders of varying size, the unit deformations within the elastic range are essentially equal.

The foregoing comment is offered as food for thought in considering the question: Why do the large cylinders show lower breaking strengths than small cylinders made of the same concrete? Does the main explanation lie in the possibility that the larger pieces of aggregate which happen to be located along the conical shear paths offer less relative resistance to ultimate failure of the test specimen as the size of cylinder is increased? Is cylinder end restraint a major or minor influence? How important are the effects of chemical heat, in causing differences in curing and internal stress conditions? It is hoped the answers will be known when the investigation has been completed and the results digested and analyzed.

The permeability tests are progressing favorably but have not advanced sufficiently to warrant the presentation of actual test data at this time. However, it may be said that the first test with an 18 by 18-in. specimen of full mix (1:9.5) mass-cured concrete made with a normal portland cement and subjected to high pressure was gratifying in that the measured permeability was materially less than expected. As a matter of speculative interest, a theoretical application of the test results to the 650-ft. base of Boulder Dam indicated that a thousand years or more would be required for water to permeate the concrete.

*See "Development of Apparatus and Technique for Measuring Elasticity of Mass Concrete," by Emile N. Vidal, in this JOURNAL.

See Editor's note top of page 9 in reference to discussion of this and other papers of this group.

Thermal Properties of Mass Concrete*

BY C. S. RIPPON AND L. J. SNYDER†

INTRODUCTION

THREE PROPERTIES, all subject to physical definitions are needed in computations having to do with the analysis of temperature movements and changes in mass concrete. They are, thermal conductivity (K), specific heat (C) and density(d). In dealing with transient temperatures these three are combined into a single abstract constant called diffusivity (h^2) which has the mathematical definition:

$$h^2 = \frac{K}{Cd}$$

Mathematical application of the theory of heat conduction and the development of special apparatus and test methods have been combined to make possible the individual experimental determination of each of the above-named properties or constants. A close check is thus obtained upon the experimental work as well as upon the theoretical applications involved.

Since artificial cooling is included in the adopted plan of construction for Boulder Dam, an accurate knowledge of the thermal properties of mass concrete is requisite to the intelligent prediction of internal temperature conditions and to the rational design and operation of an adequate and economical cooling system. Investigation of the available data on thermal properties for concrete revealed the fact that the values published were very meagre and inapplicable and original research was therefore necessary.

TEST PROGRAM

The principal object of these tests is the determination of the thermal properties of the mass concrete proposed for Boulder Dam. Aggregates from the Arizona gravel deposit are used in test specimens conforming as nearly as practicable to the concrete to be used in the dam, particularly in water content.

Supplemental tests employ materials and mixtures as used in Gibson, Ariel, Bull Run and Owyhee Dams, to determine the relation

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*Presented at the 29th Annual Convention, Chicago, Feb. 21-23, 1933. See Editor's note, top p. 9.

†Jr. Engineers, U. S. Bureau of Reclamation.

between the thermal constants obtained in the laboratory, and those computed from the records of actual temperatures at these dams during construction. The investigation includes additional tests to determine the effects on thermal properties of mass concrete, due to variations in water-cement ratios, mix proportions, cements, and mineral content of aggregates.

APPARATUS AND TEST METHODS

Thermal conductivity. Fig. 1 and 2 show the conductivity apparatus as assembled for a test of an 8 by 16-in. concrete cylinder with a 1½-in. diameter hole along the axis as indicated by "A" in Fig. 2. The ends are sealed and insulated with three inches of natural cork. The center of the specimen is filled with water and an electrical immersion heater fitted with a propeller stirrer "B" is inserted. To insure uniform water temperature inside the specimen the water is circulated at a rapid rate, which makes it necessary to apply a correction for heat of stirring. The entire cylinder assembly is immersed in a bath of tap water which remains at a constant temperature throughout the test and is rapidly circulated by a power driven propeller "C." Current is supplied to the heater and after a time constant conditions of temperature are attained at both inside and outside surfaces of the specimen. Knowing these temperatures and the rate at which energy is being dissipated through

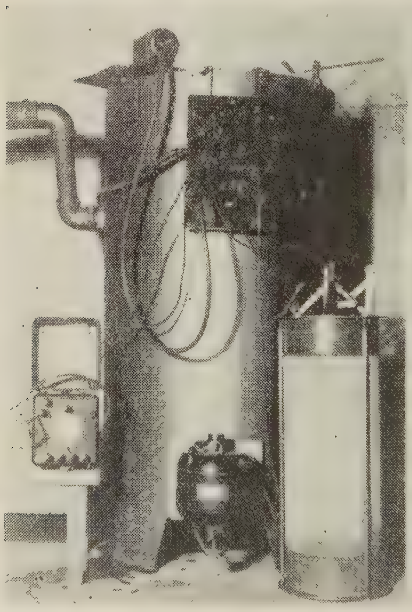

FIG. 1—APPARATUS FOR MEASURING THERMAL CONDUCTIVITY OF CONCRETE SHOWING 8 BY 16-IN. SPECIMEN WITH CORK-END INSULATORS READY FOR TESTING

FIG. 3—ADIABATIC CALORIMETER FOR MEASURING THE SPECIFIC HEAT OF CONCRETE

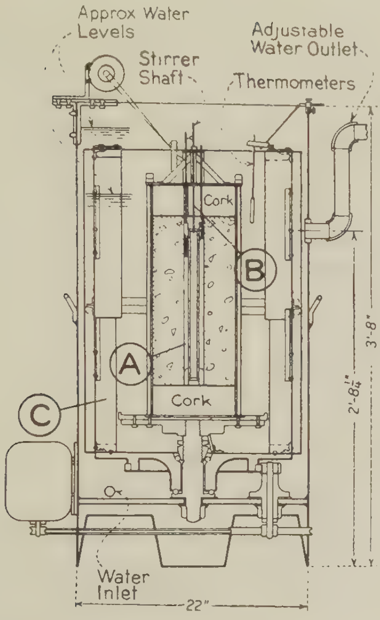

FIG. 2—CONDUCTIVITY APPRATUS

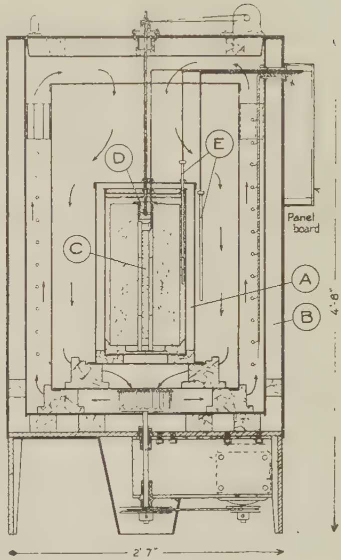

FIG. 4—ADIABATIC CALORIMETER

the walls of the specimen, as measured by an accurately calibrated ammeter and voltmeter, the thermal conductivity (K) may be computed.

Density. The water displacement method is used in making accurate density determinations A container of water is balanced on platform scales with beam reading 1/100 pound. The specimen is then suspended, fully immersed, in the water, and the increase in weight observed. The ratio of this increase in weight to the weight of a cubic foot of water is the volume of the specimen, which with the weight of the specimen in air is used to compute the density.

Diffusivity. No physical definition can be given for thermal diffusivity as may be seen from a consideration of the physical dimensions of this property. If (h^2) represents the thermal diffusivity, then

$$h^2 = \frac{K}{Cd} = \frac{\dfrac{\text{B.t.u.}}{\text{Ft. x hr. x °F.}}}{\dfrac{\text{B.t.u.}}{\text{lb. x °F.}} \text{x} \dfrac{\text{lb.}}{\text{Ft.}^3}} = \frac{\text{ft.}^2}{\text{hr.}}$$

In other words, thermal diffusivity is merely a constant which determines the *rate* of temperature change in a homogeneous isotropic material upon being heated or cooled.

The test specimen used in this determination is an 8 by 16-in. cylinder with a ½-in. round hole along the axis extending slightly below the mid-section in which a mercury thermometer is sealed. The entire assembly is placed in a bath of boiling water until constant temperatures throughout are obtained. It is then transferred to a bath of

circulating tap water of a constant temperature, and the time-temperature history at the center of the specimen observed. By referring the record thus obtained to the time-temperature curve of an ideal specimen having a diffusion constant of unity, and applying the necessary heat equations, the thermal diffusivity can be computed.

Specific heat. Determinations are made with a specially constructed adiabatic calorimeter shown in Fig. 3 and 4. The inner chamber consists of two nickel-plated copper containers, the two being separated by a one-inch dead air space "A." The outer chamber is of double wall construction with two inches of mineral wool "B" as the insulating material. The two chambers are separated by a nickel-plated copper convection shield. To insure uniform air temperature, a fan mounted at the bottom of the outer chamber circulates the air down past the inner chamber and up over heater coils mounted on the convection shield.

The test specimen, which is the same hollow cylinder used in conductivity tests, is immersed in water in the inner chamber. An electrical immersion heater "C" is inserted along the hollow portion of the cylinder, together with a small propeller "D" to agitate the water, and insure uniform water temperature. Current is applied to the heater and as the temperature of the water and the specimen rise, the temperature of the air in the outer chamber is raised accordingly. Very close control of this temperature is obtained by means of two resistance thermometers "E" one in the air, and one in the water, connected in circuit with a sensitive mirror type galvanometer. After raising the temperature approximately 70° F., the heater in the specimen is turned off and adiabatic conditions maintained until constant temperatures are noted. The heat required to raise the temperature of the specimen is then, the difference between the dissipated electrical energy as measured by a rotating watt-hour meter, plus a correction for the heat of stirring, and the heat used in raising the temperature of the water and container. This value divided by the temperature rise and the weight of the specimen gives the specific heat.

RESULTS

The performance of tests as outlined required the original design and development of special test apparatus; therefore, the results to date have been more or less preliminary in developing the desired technique required to obtain reliable and consistent results. Substitution of the constants obtained by individual tests in the relation $(h^2 = \frac{K}{Cd})$ reveals an exceptionally close check on the accuracy of the test methods and the mathematical applications involved, after consideration is given to differences in conditions of temperature under which the separate determinations are made. Experiments are now in progress to evaluate the variations of thermal constants due to temperature changes.

Fig. 5 shows the variation of thermal properties with the aggregate mineral content. The sand is composed chiefly of silica while the coarse aggregate contains only the mineral indicated in the figure. Fig. 6 gives the thermal properties of concrete representative of the dams indicated. Computations of the thermal properties from the

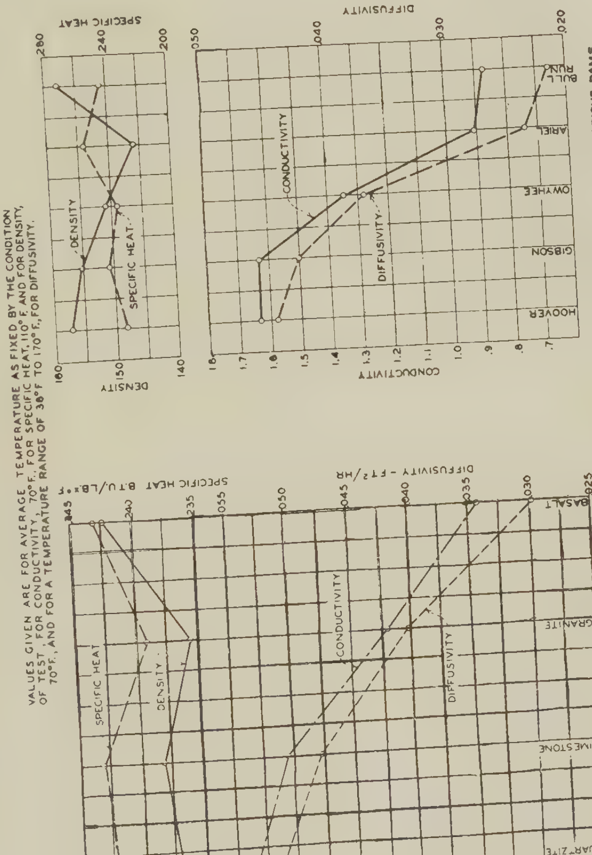

FIG. 6. AGGREGATES FROM VARIOUS DAMS

FIG 5 AGGREGATES OF VARIOUS MINERAL COMPOSITION

THERMAL PROPERTIES OF MASS CONCRETE

FIG. 5 AND 6

records of temperature movements in these dams check very closely the constants obtained in the laboratory.

From these curves it will be noted that the conductivity and diffusivity values of Ariel and Bull Run aggregates are relatively low, that Owyhee, Gibson and Boulder are relatively high, and that the Boulder Dam aggregate possesses the best thermal property values for the economical extraction of heat, the Gibson Dam aggregate which also is high in limestone content being a close second.

The agreement between the laboratory determinations and the values indicated by field measurement is most gratifying in that it insures adequate and economical design and installation of the cooling system for Boulder Dam.

See Editor's note top of page 9 in reference to discussion of this and other papers of this group.

Development of Apparatus and Technique for Measuring Elasticity of Mass Concrete*

BY EMILE N. VIDAL†

INTRODUCTION

Initiation of a program for the measurement of strains in large mass concrete test cylinders under instantaneous and sustained loading, disclosed the fact that little precedent could be found for such measurement among the multiplicity of reports on concrete elasticity tests. The rather brief researches dealing with this particular problem revealed numerous complications and difficulties, many of which were eliminated by the preliminary tests included in the program outlined by the Bureau of Reclamation. It is believed that the apparatus and technique as developed satisfy the requirements and that elasticity measurements may be made with a high degree of accuracy and with relative ease.

APPARATUS

Instruments. To establish the relation between the elastic properties of large and small specimens, apparatus was required for all sizes. After carefully studying the available literature on the subject, the five-point clamp type frame compressometer was adopted for cylinders having diameters up to and including eight inches, and individual strain gages were chosen for all larger sizes. The latter are mounted 120 deg. apart around the circumference and consist of a stiff invar steel rod firmly clamped near the top surface and loosely held in place by a suitable device near the bottom, which device also contains a milled head screw upon which bears the spindle of a dial indicator fastened rigidly to the gage rod. Thus, the total longitudinal strain over a large percentage of the height is indicated in one reading.

The extensometers for measuring lateral deformation were selected to conform to the corresponding compressometers. For the specimens eight inches in diameter or less, the clamp type was adopted. The lateral deformations in larger specimens are measured by means of separate frames, mounted on ball bearings at three points and so arranged that six dial indicators measure the deformations along three diameters, 60 deg. apart.

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*Presented at the 29th Annual Convention, Chicago, Feb. 21-23, 1933. See Editor's note, top p. 9.

†U. S. Bureau of Reclamation, Denver, Colo.

The choice of the clamp type apparatus may be questioned by those who consider the mirror type devices as being more accurate for such work. After a careful study of the two methods, it was decided that the clamp type extensometer and compressometer, if properly designed and embodying a number of improvements over apparatus of this type previously used, would furnish results having a degree of accuracy equal to that obtained with mirror instruments and would measure the deformations with far greater ease and simplicity. The apparatus was therefore designed and fabricated in the bureau's Denver laboratory with this end in view, and recent tests on drill cores from the Boulder Dam abutments using both types of instruments have furnished results with check each other very closely.

The apparatus adopted for the larger cylinders was practically a pioneer in its field. The individual longitudinal compressometers gave no trouble whatsoever after a satisfactory method of mounting was developed. Metal inserts cast in place in the specimen have proven sufficient, not only for the above compressometers but also for fastening the brackets which hold the ball-bearing supports for the lateral frames. As cast aluminum was used for these frames, because of its light weight and rigidity, its high coefficient of expansion required that relatively constant temperatures be maintained in the laboratory during the course of a test. This can be done very closely and temperature variations which seldom exceed 0.5° F. were found to have no appreciable effect on the values obtained during the period of test, usually about thirty minutes. Fig. 3 shows the apparatus mounted on a 36 by 72-in. cylinder of mass concrete with 9-in. maximum size aggregate. Note the individual compressometers and the lateral frames mounted at the quarter points. Fig. 1 and 2 are stress-strain diagrams of a test on such a cylinder thus equipped. The "curves" are straight lines and the results of the individual compressometers are in very close agreement with each other and thus with the average.

Calibration. The extreme accuracy required in these measurements made it essential that all apparatus be carefully calibrated. Inasmuch as dial indicators are the bases of all the recordings, their calibration was the first step. This was accomplished by a suitable device, especially designed for the purpose in the Denver laboratory, which provided a means of checking the dial indicator readings against Johannson gage blocks. All of the indicators were thus calibrated with readings made for increments of 0.001 in. over the full range of the dial. The clamp type apparatus was calibrated with other devices, also designed especially for the purpose and the accuracy of their

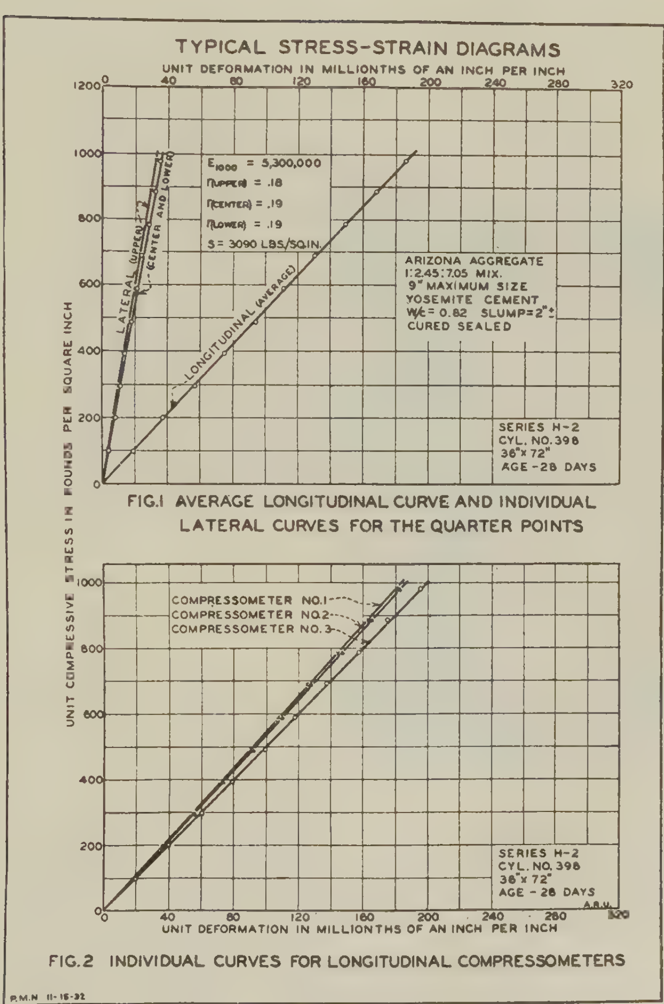
TYPICAL STRESS-STRAIN DIAGRAMS
UNIT DEFORMATION IN MILLIONTHS OF AN INCH PER INCH
UNIT COMPRESSIVE STRESS IN POUNDS PER SQUARE INCH
E_{1000} = 5,300,000
$r_{(UPPER)}$ = .18
$r_{(CENTER)}$ = .19
$r_{(LOWER)}$ = .19
S = 3090 LBS/SQ.IN.
LATERAL (UPPER)
CENTER AND LOWER
LONGITUDINAL (AVERAGE)
ARIZONA AGGREGATE
1:2.45:7.05 MIX.
9" MAXIMUM SIZE
YOSEMITE CEMENT
W/C = 0.82 SLUMP=2"±
CURED SEALED
SERIES H-2
CYL. NO. 398
36"x 72"
AGE - 28 DAYS
FIG. I AVERAGE LONGITUDINAL CURVE AND INDIVIDUAL LATERAL CURVES FOR THE QUARTER POINTS
COMPRESSOMETER NO. 1
COMPRESSOMETER NO. 2
COMPRESSOMETER NO. 3
SERIES H-2
CYL. NO. 398
36"x 72"
AGE - 28 DAYS
A.R.V.
UNIT DEFORMATION IN MILLIONTHS OF AN INCH PER INCH
FIG. 2 INDIVIDUAL CURVES FOR LONGITUDINAL COMPRESSOMETERS
P.M.N 11-15-32

FIG. 1 AND 2

fabrication is shown by the fact that of six compressometer frames all designed for a multiplication factor of two, three have exactly that factor, one has a factor of 1.98, one of 1.96, and the other (the first made) of 1.92. Fig. 4 shows the calibration instruments and the set of Johannson gage blocks.

These accurate calibrations of the dial indicators showed that there was some error present in all. However, in every case, a portion of the total range was found to be suitable for use as described below and the indicators are chosen and set so that test recordings occur within those

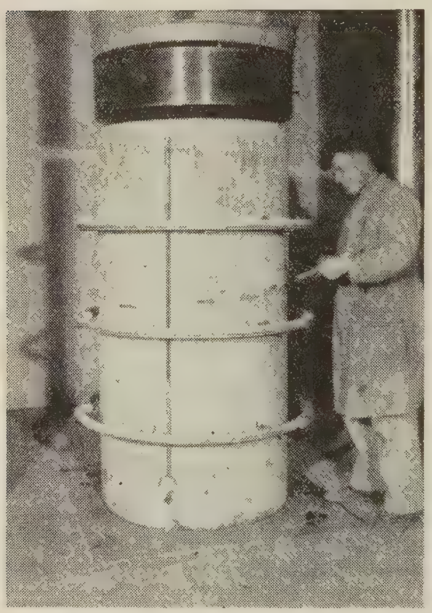

FIG. 3—36 BY 72-IN. MASS CONCRETE CYLINDER WITH APPARATUS FOR MEASURING LONGITUDINAL AND LATERAL DEFORMATIONS

limits. The calibration curves are maintained as a matter of record and check calibrations at later periods usually provide new curves, showing that the dial characteristics vary somewhat with time and age. The usual able range of the dial ordinarilly remains within the same limits, although the differences from the true readings may vary. That is, a first calibration may indicate a range from 0.010 to 0.024 wherein the dial readings are all 0.00005 inches plus or minus 0.0002 inches greater than the distances measured. Another calibration, some three months later, may then indicate a difference of 0.00009 inches plus or minus 0.0002 inches. Inasmuch as the test recordings resolved themselves into strains which are computed by the differences of dial readings for certain loads, it is seen that the plotted strains or

deformations are therefore accurate indices of the movement of the specimen under load, within the tolerance of plus or minus 0.00002 inches, a minute error and much smaller than could be plotted.

Calibration of the dial indicators only is required for the apparatus used on the larger cylinders.

All testing machines are calibrated by means of a proving-ring during the first week of every month.

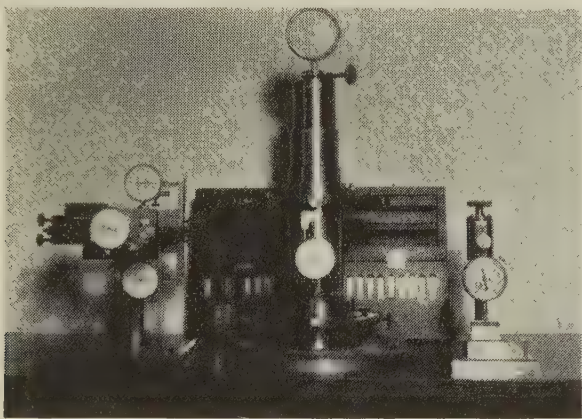

FIG. 4—CALIBRATORS FOR LATERAL FRAME, AND LONGITUDINAL FRAME AND DIAL INDICATOR WITH SET OF JOHANNSON GAGE BLOCKS IN THE BACKGROUND

TECHNIQUE OF TEST

It is almost universally agreed that a concrete specimen loaded to a maximum of about 40 per cent of its ultimate strength will provide a stress-strain diagram which is usually curved for the first application of load but is straight for the second and succeeding applications. This is due largely to lag in the instruments, imperfect end bearing, and possibly to slack in the specimen itself and to hysteresis of the testing machine. As a general rule, these effects are eliminated by first loading. Furthermore, after the first loading there is little variation in modulus of elasticity and Poisson's ratio for succeeding loadings. Inasmuch as elasticity tests in the Boulder Dam program were planned to be made with loads up to about one-fourth the ultimate strength, it was decided to preload all specimens under test to this amount and then to establish the stress-strain relation on succeeding loadings, usually two, with the first being used as a general rule for plotting.

As a means of providing uniform end conditions for all sizes of cylinders, the bureau adopted the method of surface grinding to a

smooth plane. Minute variations in this plane surface were found to have no effect on the elasticity measurements on small cylinders but variations as low as 0.004 inches in the top surface of 24 or 36-in. cylinders were found to cause erratic readings if occurring directly above a compressometer. This condition is avoided by grinding the top surface true within 0.002 inches. That eccentric loading is eliminated is seen by referring to Fig. 2 and noting again the close agreement between the individual gage lines.

For a test run, the loads are applied in increments of about 100 p. s. i. and the load is held constant while the dial indicator readings are recorded. This method was found to have no effect on the elastic properties or the compressive strength for total loads not exceeding 25 or 30 per cent of the ultimate, and greatly simplifies the observation problems.

All specimens are tested immediately after removal from the fog-room which is maintained at a constant temperature of 70° F. The measurements are made as rapidly as possible and the specimen is either tested to failure or returned to the fog-room within a maximum of three hours for large specimens. The elasticity measuring apparatus is removed before testing to failure and was designed to permit dismounting without removing the load from the test specimen.

SUMMARY

The apparatus and technique described herein are entirely satisfactory for the measurement of elastic and plastic deformations of the specimens included in the Boulder Dam concrete research program. Considerable time has been spent and innumerable precautions have been taken to insure measurements of the highest degree of accuracy. Unexpected problems have been encountered and solved and the effects of temperature and climatic variations have been eliminated or controlled to provide optimum conditions for tests. The elasticity tests made to date, both those of a preliminary nature, which were made during the course of the development of instruments and technique, and those which were made as a part of the research program, warrant a few general statements about various phases of this type of research. It should be emphasized that the following conclusions are only tentative and are based strictly on the particular conditions surrounding the tests:

(1) The stress-strain relation for stresses within 25 or 30 per cent of the ultimate strength is a straight line for all the concretes tested, including mass mixes, after being preloaded as described above and measured by apparatus suitably designed and accurately calibrated.

(2) True elastic deformations of concrete are not shown on the first loading of the specimen but are developed on the second and succeeding loadings which show very close agreement in elastic properties.

(3) Accurate calibration of dial indicators and measuring apparatus based on dial indicators must be obtained periodically to insure accurate measurements by this type of instrument.

(4) It is essential that uniform bearing be provided between the testing machine and test specimen. Eccentric or point bearing has considerable effect on the elastic deformations under load and, if present to only a small degree, will destroy the value of their measurement.

(5) With the apparatus as designed and adopted, the elastic deformations of concrete test specimens up to 36 in. in diameter may be measured as accurately as those of small specimens and with relatively small increases in time and effort.

See Editor's note top of page 9 in reference to discussion of this and other papers of this group.

VIBRATION AS A MEANS OF PLACING CONCRETE

At the Institute's 29th Annual Convention, Chicago, February 21-23, 1933, considerable time and attention were given to the subject of vibration as a means of placing concrete. Four papers in the group were published in the JOURNAL *for June 1933, Proceedings, Vol. 29, pp. 365-396: "Compaction of Concrete through the Use of Vibratory Tampers," by Raymond E. Davis and Harmer E. Davis; "Vibrated Concrete," by T. C. Powers; "The Use of Vibration in the Manufacture of Concrete Products," by Miles N. Clair; "Vibratory Finishing Machine for Concrete Pavements," by F. V. Reagel. Six other brief contributions to the series, supplemented by the convention discussion of the subject, appear in the following pages. For such further discussion as may develop, readers are referred to the* JOURNAL *for March-April, 1934. Such discussion should be available to the Secretary of the Institute on or before February 1, 1934. The attention of interested readers is again directed to the fact that the paper by Davis and Davis and the one by Powers were published in summary only and that complete copies are available to* JOURNAL *readers—the Davis paper to members at 50 cents, to non-members at $1.00 per copy; the Powers paper at 75 cents to members, $1.25 to non-members.*—EDITOR

High Frequency Vibratory Machines for Concrete Placement*

BY M. I. MCCARTY†

While numerous data have been presented to you on results accomplished with vibration in placing concrete, something should be said concerning the mechanics of the method. Webster's definition of vibration is that of an action produced by practically anything from a tack hammer to an earthquake. Obviously the successful use of vibration in placing concrete lies in the ability of the vibratory machine or machines to place and make homogeneous, mixes of a lower workability factor than may be possible or economical with the usual hand spading methods. Seven years ago we seriously undertook the task of investigating the possibility of introducing vibratory waves or force to semi-plastic mixtures, not with the idea of solving any individual problem but rather for the development of a practical process or method of general application. Having no previously developed data of value upon which to base our experiments, we felt our way carefully, meeting problems one by one and working them out under field conditions. Laboratory results have from the first been highly satisfactory with but few exceptions, indicating the advantages of harsher mixes provided they could be properly puddled and keeping in mind that surface appearance is not always indicative of strength, density or other desirable concrete qualities. It has been and is our desire to make equipment available to meet the demand for improved placing methods. We have had the helpful assistance of the Portland Cement Association, the Bureau of Public Roads, the Bureau of Standards and other Government and State institutions.

Three mechanical factors in the vibration of concrete are of utmost importance if the full benefits are to be realized: frequency, amplitude or force of the action and use of the correct machines for the job in hand.

*Presented at the 29th Annual Convention, Chicago, Feb. 21-23, 1933.
†Electric Tamper & Equipment Co., Ludington, Mich.

Early in our investigations we decided that a revolving element or unbalanced member produced the desired vibratory wave, affecting the mass for a considerable distance. The next step was the determination of the correct frequencies and weight of the unbalanced member. It soon became apparent that speeds lower than 3000 r. p. m. were inefficient, that of 3600 r. p. m. obtained with a two-pole electric motor operating from a 60 cycle power source was satisfactory and economical. Later investigation revealed that greater vibratory efficiency is obtained in frequencies above 4000, the measure of efficiency being the time required to puddle a given mass completely without separation. Frequencies in excess of 4500 r. p. m. are not now mechanically practical in a rugged machine to withstand the abuse to which such equipment is subjected.

One other objection to extreme frequencies lies in the necessity of reducing the weight of the unbalanced or vibratory member and by so doing correspondingly reducing the amplitude to the point where it becomes ineffective in handling concrete in any quantity. For light castings in the products division of the industry the reduced amplitude may be desirable and for this purpose we have produced a vibrator in which amplitude may be adjusted quickly from zero up.

A machine designed for placing concrete in a deck slab would not answer the requirements for use on a dam; one used successfully for column encasement work would not be applicable in the cast stone industry. Internal vibrators are in many instances far more efficient and economical than external vibrators and while the principle involved is identical the successful application is worthy of consideration.

Various machines have been designed to meet various problems. It is necessary only to change the attachments on the external vibrator for various operations. Our first machine for vibrating concrete was a motor, of the type now used, mounted on a sled. The sled was in reality a piece of sheet metal bent to permit its being pulled over the surface of the concrete. Such equipment was used in cooperation with the Denver Tramway Co. in 1925 and from this early work a finishing machine was developed that is now in use.

Fig. 1 shows a puddler built in 1927 and placed in service for the Aluminum Co. of America at Calderwood, Tennessee—a piece of equipment to assist in placing mass concrete in large areas, light enough to be handled by two men and designed to handle harsh mixes. The unit consists of a motor with unbalanced weight, flexibly connected to a handle for manipulation, mounted on a float 18 x 36 in. Such a machine subjects the concrete to vibration under compression and

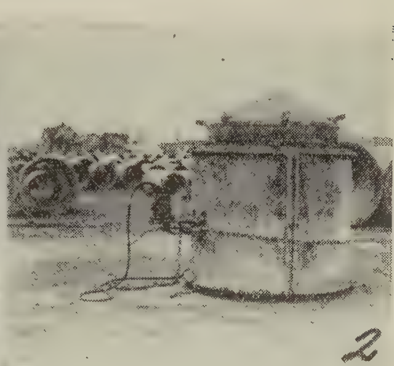

FIG. 1—A PUDDLER USED ON MASS CONCRETE IN 1923

FIG. 2—VIBRATOR ATTACHED BY CHAIN TO PIPE MOLD

FIG. 3—SPUD VIBRATOR ON SLOPING ROOF

FIG. 4—VIBRATOR MOUNTED ON FLOAT

FIG. 5—INTERNAL VIBRATOR

frequencies in excess of 3,000 r. p. m. are essential. On the Calderwood project two of these machines, each one operated by two men successfully puddled into place four yards of harsh concrete in from three to four minutes. If means were available to handle much heavier equipment it is possible a unit weighing as much as one ton would prove efficient.

Fig. 2 shows a machine designed for casting concrete pipe—an entirely different problem since the vibration is applied to the form. It was at once apparent that if the vibratory action were to be effective around the entire circumference of the form the machine could not be attached rigidly to the form in one spot. An encircling chain transmits vibrations to the entire form, successfully placing concrete of extremely low water ratio.

Equipment like that used in placing concrete for pipe is attached to the forms for reinforced concrete columns, as on the Ogden Avenue grade separation in Chicago. Similar equipment was also used on the heavy columns of the Schulykill River bridge in Philadelphia on which a vise attachment was used. This same vibrating motor attached by means of a vise was used in placing concrete in the counterforts on the Hamilton Dam in Texas. The number of machines necessary depends entirely upon the thickness of the wall and the consistency of the concrete.

Vibration may be applied through the form to the concrete by what is called a spud machine, consisting of a vibratory motor with a flexibly connected handle, the motor being bolted to a 2 x 4 of suitable length for the conditions of the job. The end of the spud, in contact with the bottom form, transmits the vibration through the spud to the form and thus to the concrete. With such equipment concrete was placed on the deck of the Schulykill river bridge in Philadelphia. On such work the vibration is noticeable 20 to 30 feet from the machine. The blow itself is not severe and the action therefore is not hard on the form work. Similar equipment was used in placing concrete under and around deep I-beams on the railway grade separation job at 16th and Canal streets, Chicago, and on other similar work.

Fig. 3 shows an unusual application of the spud vibrator where the concrete was placed on a sloping roof. It must be obvious that the concrete was very dry, otherwise it would immediately have flowed out of position, yet the resulting under surfaces of this work were excellent.

In connection with placing Truscon and Carnegie types of grid decks where a spud machine has ordinarily been used, we recently employed successfully what we call a hand screeding machine. The

work shown in Fig. 4 was at St. Joseph, Mich., where a vibrator was mounted on a 6-ft. float or screed having flexible handle. On this job ½-in. slump concrete was placed readily, struck off 1 in. above the metal. This type of placement may also be accomplished with the use of a spud vibrator.

Fig. 5 shows an internal vibrator—a vibrating spade. The vibratory element is attached to the lower end of a manipulating handle by means of a flexible connection which prevents the vibration from affecting the operator. The diameter of this machine is 6 in.—it may therefore be used in walls 10 in. or more thick. The handle may be lengthened as desired.

It is demonstrating its value in mass work where the motor is submerged from one to three feet within the concrete causing it to flow readily into position. The machine is easy of manipulation with a control switch at the end of the handle and produces a smoother and more dense surface than may be obtained with the external or form vibrator. Furthermore the action of the machine is directed to the center or heart of the mass where complete consolidation is most important. We think it is the most efficient and economical tool for the construction industry we have developed.

One of the latest applications of vibration is that in which the action is used in connection with the finishing machine for highway paving reported in a paper by Mr. Reagel of the Missouri State Highway Department.

In reference to discussion of this and other contributions to the subject of "Vibration as Means of Placing Concrete" see Editor's note, page 48.

Placement of Concrete by Mechanical Vibration*

BY A. W. MUNSELL†

Present knowledge of the art of concrete making favors minimum water for uniformity in the quality of the product. This has made placing difficult by ordinary methods if honey-combing, stone-pockets and unsightly surfaces are to be avoided. To overcome this condition various methods, including manual and mechanical equipment, have been used for vibrating the forms without much investigation into the effectiveness of such equipment.

The writer's first experience with the use of vibration was in placing concrete for the hulls of concrete ships. A burnt clay aggregate with the harshness characteristic of this material made placement extra difficult in spite of the diatomaceous earth added for lubrication. There was difficulty in making the concrete flow around the three lines of closely spaced reinforcing steel. It was noticed immediately that the concrete was not flowing and 40 carpenters with their hand hammers were placed at the point of pouring to hammer the forms, this was ineffective because of a lack of harmony in the blows, besides being too expensive for labor. Later 20 small air driven chipping hammers (with frequencies of 7000 blows per minute) were used with better but not satisfactory results.

The writer's next experience with the use of vibration was in connection with placing deck concrete on the Delaware River bridge at Philadelphia, where various types of equipment, both pneumatic and electric, were tried out. The pneumatic tools were of the usual piston type with a travel of ¾ in. to 7 in. and with frequencies from 1700 to 7000 per minute. The longer piston travel damaged the forms and displaced the reinforcement, while the higher frequencies with a shorter travel had a tendency to smooth out and become ineffective. The electric equipment was of the unbalanced rotor type with frequencies of 3000 to 4000 per minute. The eccentricity of the rotor was approimately ½ in. From these experiments an air tool with a piston stroke of ¾ in. and a frequency of approximately 3000 per minute, gave results most satisfactory among tools then available.

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Studies of the essential requirements for such a piece of equipment, for use on the deck concrete of the George Washington bridge over the Hudson river, the Bayonne bridge over Kill van Kull and other structures under the Port of New York Authority, showed that four main factors should be considered in the design of such a tool. (1) the speed or frequency, (2) control of the frequency, (3) the amplitude of the blow, and (4) the time of application.

The consistency of the concrete to be placed would be the controlling factor in the frequency to be used, for wet (4 to 6 in. slump) consistencies a low (1000 to 1500) frequency, for dry (1 in. slump) consistencies higher (3000 to 4500) frequencies should be used, also depending on the amount of water used and the mass to be moved.

Control of frequency is important in taking care of changes of consistency during a run of conrete. This occurs very often in placing beams and slabs in one operation, the concrete for the beams usually is much wetter than that required for flat slabs, therefore, when a change of water content is made in the concrete a change of frequency should be obtainable in the vibrating equipment. This is also true of high walls where the water gain appears on top of the concrete during a run.

The amplitude of the blow has a bearing on the type and ruggedness of the forms to be used. A low amplitude (¼ to ⅜ in.) permits the use of comparatively light and economical forms and a light blow will not displace the reinforcing steel,

The time of application will depend on the consistency of the concrete and the type of structure in which this method is used; for dry concrete in floors or bridge slabs (6 to 10 in deep) where the upper surface is subject to wear the application should not be more than two minutes for each 10 sq. ft. of area, and the application should be on the forms to minimize the effect of such vibration bringing the fine material to the top and making a less durable wearing surface. In high walls the application should be carefully timed because if applied too long (5 minutes) there is a tendency to force the fine material away from the point of application and bring the large aggregate to the surface, which gives a non-uniform texture when the forms are removed. It is good practice to move the vibrators up as the wall is filled, for instance, in placing concrete in a 12- to 18-in. wall the vibrators should be moved upward every three feet of height. It has also been observed that too long an application from a vibrator rigidly fastened to the form tends to bring air and water bubbles to the inside face of the form and leave pits and holes similar to inverted blisters.

Contrary to some opinions, I do not believe that vibration makes stronger concrete. That is entirely a function of the amount of water used. The water-cement ratio will fix the strength as has been proven to the satisfaction of all those who have followed the various reports that have been made on that subject in the last several years. Vibration, however, aids and hastens densification. This consolidation has the effect of shrinking the concrete immediately by liberating the entrained air and water and probably quickens the initial hardening of the concrete by leaving only the minimum of water required for the chemical reaction. A few tests were conducted in the laboratory of the Port Authority which indicated that mechanical vibration increased the weight of concrete used from 5 to 10 percent depending on the kind of aggregate used. It will also permit the use of more economical mixes because of the easier placing, and will minimize patching.

Vibration reaches corners and spaces in closely spaced reinforcing that can not be tamped and rodded and gives assurance that such steel is completely imbedded.

Bond strength, as shown by the very complete series of tests conducted by the U. S. Bureau of Public Roads in cooperation with the Port of New York Authority, reported in the December 1931 issue of *Public Roads,* is markedly greater for vibrated concrete on both plain and deformed bars.

Because of a lack of test data on this method of placement our opinion of its merits must be based on the superficial observation of the results obtained. Mechanical vibration is a natural step forward in the modern practice of concrete placement made difficult by limitations on water-cement ratio. It is my opinion that mechanical vibration will result in a more uniform quality and more durable concrete and should be made to pay for itself by minimizing patching and rubbing costs rather than by a too economical use of cement and aggregates.

In reference to discussion of this and other contributions to the subject of "Vibration as a Means of Placing Concrete" see Editor's note, page 48.

Vibration on Michigan Bridge Work*

BY A. C. BENKELMAN†

WHILE OUR Department had planned to use a vibratory finisher in placing about two miles of concrete pavement in 1932 because of the lateness of the season this work was postponed. We are not, therefore, in a position to take part in a discussion of this method of finishing concrete pavements at this time.

The experience which the Department has had with the use of vibration in bridge work may be briefly summarized as follows:

We feel that the vibration of concrete during placing has several advantages which, to date, are being borne out on jobs where we had opportunity to try it out. We are requiring the vibration of concrete on all of our jobs now being contracted. At Manistee on the concrete work for a bascule bridge, all concrete is being vibrated, including the concrete piles which were precast for a part of the substructure. We are making of the Manistee bridge an experimental study in the efficiency and value of vibration and also a study of the comparative efficiencies of different types of vibrating equipment.

While all of this equipment is produced by the Ludington Electric Tamper & Equipment Co., we are trying out really three types. One is a vibrating motor enclosed in an elliptical shell, mounted at the end of a long pipe which serves as a handle and permits the vibrator to be operated in wall forms and mass concrete from a point at any reasonable height up to about 20 ft. above the point of application. This vibrator is designed to operate in the concrete without contact with either forms or reinforcement and it is readily controlled and moved around.

The second type of vibrator is very similar to the air or pneumatic vibrators in that it is applied in shallow casts, such as the concrete piles particularly, and consists of a tamper head of approximately 6 in. on a side operated very similarly to the handle of a jack hammer; this so-called spade is shoved into the concrete and is most effective when pressed against reinforcing steel in the cast or against the form itself, although it is effective even in the concrete area alone.

The third type is similar to the second except that the vibrator acts through a plank or similar large surface designed to rest on the top of forms, as in the case of floor forms, or even on the top of reinforcing steel mats; this equipment being varied in some instances by mounting it on a car which enables the large floor area to be handled systematically and easily.

In testing these three types we find that it has been possible to compact, free from honeycomb in a concrete pile, using type number two with a concrete having a

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†Formerly Research Engineer, Michigan State Highway Dept., Lansing.

slump as small as ½ in., although the work required for such a product is so excessive as to make this impractical. We find that the number two equipment, using a slump of about 1½ in., is very efficient and highly satisfactory and with our grade A proportions will develop driving strength in three or four days time. We find, however, that great care is needed in the use of this spade to see that areas are not skipped over during the vibrating process, as in such cases the concrete will be much poorer than would have been secured by older methods. To overcome this we found it necessary to spike cleats over the forms at about 4 ft. spacings and insist that the vibrator operate in each cell successively.

We tried the equipment described as number one in mass concrete and find that it is necessary, for best results, to vibrate against the reinforcing steel in the forms. We also tried this equipment in a tremie box placed about 4 ft. under water, the box being filled with a tremie spout and the concrete vibrated at the same time with this equipment. It was felt that very satisfactory results were secured. We have not yet tried it out in the casting of the tremie seals for the piers on this job, but hope to have some idea of its value before the work is complete.

We have not yet tried out the type number three equipment which will be used on our floors and other intermediate decks. The tendency of vibrating equipment to bring up air and water at the sides of forms, while undoubtedly greater than with puddled concrete, we find is not of sufficient importance to effect the finishing operations for a very satisfactory rubbed finish and the freedom from honeycomb secured more than offsets this difficulty in any event. The fact that cost is as low as $1.00 per day for a vibrating machine, together with the fact that with ordinary small pours one machine can keep up with a casting, makes it appear that vibrating concrete is not only more satisfactory but is also more economical than the old puddling methods. The claims that the amount of cement used in the concrete can be reduced, due to the fact that you can use a drier mix and therefore get higher strength with less cement, does not seem to be borne out by our test beams and cylinders on this work as the ultimate or twenty-eight day strength of our concrete is not materially different from that secured with puddling methods and the same proportions. We feel, therefore, that considerable care or judgment should be exercised in proportioning modifications.

The additional cost to the contractor of more rigid forms and bracing, will likely disappear with the improved types of vibrators which work through the reinforcing steel rather than through the forms themselves and in locations where rigid forms and bracing are provided by the specifications, it is believed that but little, if any, increased cost to the contractor from this cause will result. Conservatism should be shown in selecting form ties of reasonable diameter as the vibration is very likely to snap rods of the smaller sizes in which stresses are usually much higher than good design would dictate as safe.

We feel that there is a broad field open for development of vibrating equipment which will be handled from inside the forms by bringing the vibrator in contact with the reinforcing steel rather than applying the vibrator through the forms themselves.

In reference to discussion of this and other contributions to the subject of "Vibration as a Means of Placing Concrete" see Editor's note, page 48.

Vibrating Equipment in a Cast Stone Plant*

BY GEORGE B. PICKOP†

At the cast stone plant of the Dextone Decorative Stone Co., New Haven, Conn., extensive use is being made of several types of high-frequency vibrating equipment, in different classes of work.

Molds of both wood and plaster are employed extensively, while gelatine molds are used for intricate work involving undercuts and other difficult details. All of these molds are designed to be rigid enough to withstand the action of vibrators. Sand molds, likewise, are employed for some work.

The proportioning of aggregates, cement, and water at the Dextone plant involves no special problems, but several distinct advantages in proportioning are obtained as a result of the use of high-frequency vibrators. To begin with, since the vibrators have been installed the proportion of fines has been reduced considerably. This is a step in the right direction, since it is now well known that an excess of fines may result in crazing. In the second place, the use of the vibrators permits the use of a lower water-cement ratio and a drier consistency than were formerly possible; and this, again, is a step in the direction of increased strength. In a word the mixtures can now be proportioned in accordance with the known principles of concrete design, with the view of obtaining maximum strength and density.

Roughly, it can be said that with the use of vibrators the strengths average about 100 per cent higher than formerly, while the absorption has been reduced from 7 to 3 per cent, by weight.

Still another advantage in the use of vibrators comes from the wider scope in texture, due to the fact that higher percentages of coarse aggregate can be employed in the mixtures. Vibration consolidates masses of relatively dry concrete, with relatively large proportions of coarse aggregate, in a manner that was impossible when ordinary puddling methods or the flow of the material had to be depended upon to fill the molds. This permits close approaches to cut or polished natural stone, since so much of the surface is actual stone aggregate. These products can be polished in 3 days, while before the vibrators

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†Asst. Superintendent, Malleable Iron Fittings Co., Branford, Conn.

were installed it was necessary to wait from 7 to 14 days before mechanical polishing was attempted.

The Dextone plant employs several manufacturing processes in its cast stone department. One is the hand or floor molding, the impression of the object being taken in the molding sand that is built up on the floor. Since it is impossible to vibrate the floor, a hand spade of the pneumatic vibrator type is inserted into the freshly placed concrete and put into action. The spade is held at an angle of 15 to 45 deg. with the vertical, the angle depending on the depth of the concrete in the mold. The vibrating action brings laitance to the surface, and this overflows into the sand.

Molds of wood, plaster or gelatine are usually filled on a vibrator table. On some of the work one large high-speed vibrator is mounted at the center of the table, which in turn is mounted on springs in its understructure; or the table is equipped with four small vibrators, one near each corner. This arrangement of four small vibrators, operating with a frequency of 9,000 to 10,000 vibrations per minute, is especially effective where the molds contain from 1 to 2 cu. ft. of concrete.

A slicing spade of the pneumatic vibrator type, the vibrating part being about 4 ft. long, was primarily designed for placing structural concrete in columns, building walls, bridge abutments, and similar locations where the material is placed in stages several feet deep. This has been useful in vibrating large units, cast in up-ended molds, and it is used similarly for vibrating large concrete tile.

Experience at this plant has demonstrated the value of one feature of the Branford vibrating equipment employed, namely, its variable speed. It has been found best to operate the equipment at an idling speed (anywhere from 200 to 1,000 vibrations per minute) while the concrete is being deposited. Then, when the mold is filled, a short spurt of high-frequency vibration completes the work.

It has been found also that dry mixtures require considerably longer periods of vibration than the more plastic mixtures. The plastic mixtures, of course, are employed in the more complicated molds, where it is necessary to vibrate the concrete into the more intricate details of the mold. The drier consistencies are used, as previously intimated, for blocks of cast stone that are to be polished.

Branford vibrators can be operated by any air compressor equipment capable of delivering approximately 15 cu. ft. of compressed air per minute at a pressure of 60 to 80 p. s. i.

In reference to discussion of this and other contributions to the subject of "Vibration as a Means of Placing Concrete" see Editor's note, page 48.

Fabricating 36-in. Reinforced Concrete-Steel Cylinder Water Mains*

BY J. F. BRETT†

Work in progress by the Montreal Water Board consists of the fabrication of approximately 60,000 ft. of straight pipe and specials suitable for city water works' purpose and designed for a working pressure of 120 p. s. i. Pipes consist of an outside shell 2¼ in. thick placed with vibrators; the inside shell 1¼ in. thick is placed by centrifugation. The pipes are 16 ft. 6 in. long and weigh 5 tons each.

The specifications call for concrete having an ultimate compressive strength of not less than 4,500 p. s. i. at 28 days, when tested in 6 by 12-in. cylinders. The aggregates are sand and crushed limestone, specially graded for the purpose.

After some weeks of experimenting at the pipe company's plant, very good results were obtained with a mix by volume, of 1:1.6:1.9; fineness modulus of sand from 3 to 3.2, that of the coarse aggregates from 5 to 5.4. Total water is about 35 lbs. per cu. ft. of cement. The mixing time in a vertical paddle wheel open type mixer is 3 to 4 minutes. These pipes are cast vertically in steel molds. Four air vibrators are attached to the forms and kept going continuously during the pouring which is also continuous.

During the early stages of manufacturing, some trouble was experienced with shallow voids and depressions on the outer surfaces, caused either by water or air bubbles. This was completely eliminated by using larger vibrators. The concrete in the outer shell varies in strength from 5,600 to 6,500 p. s. i. at 28 days, for 6 by 12-in. cylinders cured in the laboratory.

The French claim our rotating vibrators are rather wasteful of energy, but I think, outside of that, results obtained with the reciprocating models do not differ from what we are getting here.

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*From a letter in response to inquiry of Mr. Brett, written as a contribution to the consideration of experiences in vibrating concrete at the 29th Annual Convention, Chicago, Feb. 21-23, 1933, and in the author's absence was not presented.

†Montreal Water Board, Montreal, Quebec.

We made an experiment in placing the outside lining of 36-in. concrete steel cylinder pressure pipes. Reciprocating air vibrators (Société de l'Outillage Pneumatique de Paris) have been used on this work, both on our first contract in 1930-31 and again this year. The Jackson machines were tried, bolted to the molds, with same mix and consistency, and no difference could be seen in the results.

Our experience with vibrators since we used them on a 150-ft. double spiral boom bowstring girder bridge in 1930, is that no difficulty should be experienced with concrete showing a slump of 2 in. or less provided forms are reasonably tight. We use them now on all our concrete work, precast or cast-in-place. The vibration is stopped as soon as laitance begins to appear, this point can only be detected correctly when no water lies on the surface. The concrete should be vibrated thoroughly, this is probably overlooked sometimes; it is easy to overestimate the effect of the vibrators on the concrete when they are attached to the forms which absorb so much energy. An excess of paste tends to hold air bubbles.

We are following closely the making of our pipes referred to above. About twenty 16-ft. pipes are made daily, and many experiments are being carried out.

In reference to discussion of this and other contributions to the subject of "Vibration as Means of Placing Concrete" see Editor's note, page 48.

Vibration In Making Roof Deck Slabs*

BY A. B. SHENK†

For a number of years we have been producing, in steel molds, precast slabs of concrete for roof deck purposes. Until a few years ago the concrete was puddled into place and the molds jarred by pounding or rattling by hand. A good quality of concrete was obtained and the pounding or jarring provided a finish on the underside of the slabs where the concrete was in contact with the steel forms.

Hand labor was replaced by hanging on the form a pneumatic vibrator and this experiment proved that the concrete could be worked into place and a proper density and compactness obtained by means of this vibration without resorting to puddling and pounding or jarring. Eliminating hand labor thus speeded up the operation.

At the same time we found that we were getting concrete just as strong as by the previous method with a good finish on the back of the slabs where the concrete came in contact with the steel.

As time went on it developed that by means of this vibration equipment we could produce this type of slab, securing satisfactory filling of the molds, and satisfactory finish on the underside, at the same time using a concrete of lower water ratio.

The mix has a slump of approximately two inches, as determined by the standard slump cone test. In all of our work we have used a pneumatic type of vibrator having an unbalanced piston driven back and forth by the supply of air. In the type we use the piston diameter is approximately one inch and two of these are hung on the steel form which weighs about 300 lb.

In the course of developing vibration for our own use we found that there was a difference in effect produced by the amplitude of vibration and its speed. Seemingly the greater the amplitude of vibration the more rapidly the concrete flowed into position in the form. On the other hand the greater speed seemed to produce a slower movement of concrete into place but with a corresponding increase in the density

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of the finished concrete. The equipment which we are using in the size described and with a speed of around 2000 vibrations per minute seems to strike the proper balance between speed of placing and degree of density for our particular purpose.

Our experience has shown that the use of vibration facilitates manufacture and leads to improved quality both of concrete and of the articles manufactured therefrom. We consider it a practical, economical, and reliable method of placing concrete under our conditions.

In reference to discussion of this and other contributions to the subject of "Vibration as a Means of Placing Concrete" see Editor's note, page 48.

Vibration As a Means of Placing Concrete*

CONVENTION DISCUSSION*

Alfred E. Lindau (*Chicago*): To the contractor, the man out in the field, many of the problems presented here, problems that will probably find solutions in change of equipment, have no particular meaning. The man in the field has so much concrete to place and the quicker and easier he places that concrete, the better he is satisfied. We have not attempted to see what the full possibilities of the machines may be in connection with variation of mix and change of aggregates, economy of cement, etc., excepting in some instances, one of which was reported by Mr. Powers, and formed the basis of his discussion this afternoon. As I see the situation from the very practical standpoint of getting concrete in place, it looks to me very simple, but of course it is not. You have concrete to get in place. You want to place it without segregation, so that you don't have to come back and patch it. So the vibrator, as we see it, is a means of giving mobility or action to the concrete. We want to make that concrete flow into place and fill up all of the form space. The vibrators we have used seem to perform that function with various degrees of success depending upon the kind of mix we have. In practice, it seems of no use to vibrate very wet mixes—with seven, eight and nine inch slumps, we may just as well forget the vibration. It is such slumps which characterize what the contractor calls his "pour." When concrete pours there is not much the vibrator can do. My interest in the vibrator from a technical standpoint arises from this general consideration, and it has been confirmed in going out on the job. We have spent time, money and effort in attempting to get a concrete used with a low water content. I see little effect of all that effort as I go out on the job. I have seen some very fine concrete and I have seen in the general run, however, concrete that you could not do very much with. As an engineer told me the other day, we spend a great deal of time with the design of concrete, the design of the structure, and when we go out on the job and see what

*Convention discussion here recorded in brief concluded a special session on Vibrated Concrete at the 29th Annual Convention, Chicago, Feb. 21-23, 1933. Four of the papers were published in the JOURNAL for June 1933; *Proceedings*, Vol. 29. p 365-396. See Editor's note p. 48.

they do with it, we are very much disappointed. If this vibrating mechanism, whatever type may finally be used, can put the water-cement ratio into effect so you can get into the form the kind of concrete you design, then we have accomplished a very great deal. I do not pretend to have had sufficient experience to speak of various types of vibrators. They are separated, in my mind, into two main classes, those vibrating in the concrete, and those that vibrate from the outside. Both have their uses and their disadvantages, and as to where one is more advantageous than the other, I am not prepared to say. We need more data of the kind we have had this afternoon. One of the things that has occurred to me frequently, and we have seen it in the operation of these vibrators is, that the tendency of concrete is to have the fines in the concrete flow toward the point of vibration. Take the internal vibrator and stick it in a mass of concrete and leave it long enough and the mix liquefies around that point. I assume the same thing happens if you vibrate externally. For many reasons it is not desirable to have the fines at the surface.

I wish to refer particularly to one instance of economy like that cited by Mr. Powers. I hesitate to talk about the cement you can save, as though you are trying to save cement, it is not the only thing to be saved, but if you can make a legitimate saving, without skinning the job, without sacrifice of strength or durability, then I think that saving is worth talking about. It may even be possible to increase durability while saving cement. An internal vibrator was used in a sewage disposal plant in Wisconsin; the contractor was able to use five sacks to the yard instead of six as estimated, and if the engineer had permitted he could have reduced the cement to four and a half sacks per cu. yd. without increasing the difficulty of putting the concrete in place or having to do any patching. That is one of the things that can be done. The saving in placing is not always apparent, it depends upon the job; we have seen jobs where vibration did not effect a saving in placing where there were surplus men on the job who had to be carried. In other instances distinct savings could be made. On one job in Milwaukee five men using sticks were replaced by one man with a vibrator at a very big saving. I wish to mention two other items, the subject of frequent questions. One is, why do you want to run the vibrator 5000 r. p. m.? Certainly it is not an advantage to run it that fast, the bearings wear out, the equipment is more expensive and there are a great many mechanical disadvantages to so high a rate of vibration. Well, we find when we have a power condition which cuts down the rate of vibration, we do not get as much work done. I do not know

where the limit is, whether it is six, seven, eight or nine thousand so far as output is concerned, but you will get more work done in the way of putting concrete on the job with a high than a low rate of vibration.

Another question is, what does the vibration do with the forms? Professor Davis has answered that. It arises all the time: "Do you increase the pressure on the forms?" I have been loath to give to the contractor an answer to that because apparently at times it seems as though you did. My own idea is that you can keep a larger volume of concrete fluid by your vibration and increase the hydrostatic pressure and may have more pressure on your forms.

Harlan H. Edwards (*Claremont, Cal.*): We used the internal type of vibrator for the work in California last year, placing walls 4 to 8 in. thick, and slabs from 2½ to 6 in. thick, very well. We made savings of from half a sack to a little bit better than a sack, our lowest cement proportions being about four and three quarter sacks per yard, and yet secured concrete that tested two to three thousand pounds, depending on conditions.

In the figures presented by Professor Davis this afternoon, it occurred to me particularly to question whether the bond which he obtained between new concrete and old concrete would be sufficient to withstand shearing stress between the new concrete and the old, at a later period when the newer concrete in drying and shrinking, is taking its final position? In general shrinkage cracks do not show for a year or perhaps a year and a half after the concrete is placed, then, due to temperature movement and shrinkage from one reason or another, they do appear. Possibly this would suggest the need of tests along that line. In bonding a concrete which has hardened and has taken a good deal of its shrinkage, to another which is fresh and going to shrink later such shearing stresses are certain to occur.

In building work, the greatest difficulty I have had in the use of the vibrator seems to be with the forms for exposed concrete. If we can assure ourselves of a tight form we have very little trouble. If we are a little careless or cannot afford the cost of absolutely tight forms, we sometimes get sand streaks in the finished product at the form's joints, lines difficult to remove. I have found such streaks can be largely eliminated by following the vibrator with one man using a metal spade going up and down, working the entire exterior of the wall to break up the flow lines, of excess water escaping through the joints.

Aside from the concrete itself, we have problems of consolidating back fill in certain types of soil after we have placed a structure, and recently I have used the internal type of vibrator with a little water to

help in the consolidation of back fill of granular material such as sand and gravel, perhaps decomposed granite, which composes the soil in some districts, and have found that I can consolidate that material better than with water alone.

Finally, in connection with the Pine Canyon Dam, I was out there recently to see the work of the different types of vibrators. It was a revelation to see the way 1-in. slump concrete was handled. It was dumped from 4-yd. hoppers, spread out a little and then worked over by the platform type of vibrator. The minute a spud or internal vibrator got into it you could see the concrete change character. It would settle down, flatten out, perhaps flow—taking on a liquid appearance in its consolidation.

A Member: Has there been any study of the possibility that the permeability of concrete will be increased in leaner mixes encouraged by vibration?

T. C. Powers (*Chicago*)*:* We have made no permeability tests of vibrated concrete. We have, however, some indirect evidence from the freezing and thawing tests. These show that the extreme mixes placed by vibration, very lean in cement but with low water-cement ratios, withstand freezing and thawing fully as well as richer mixes of the same water-cement ratio, placed by hand. This is to be expected, for just as in workable mixes placed by hand, each and every aggregate particle in vibrated concrete is surrounded by mortar, although it may be a thinner layer of mortar than in ordinary mixes. This means that just as in workable hand-placed mixes the only possible path which the water may follow through the mass is through the matrix in which the aggregate lies. It is apparent that water-tightness depends on the quality rather than the quantity of this matrix.

E. A. Hagy (*Cincinnati*)*:* As I see it, especially in paving concrete, the idea of vibration is either to reduce cement quantity per cubic yard or raise the strength of the resulting concrete. Have any studies been made as to what is the direct relation of strength to durability or cement factor to durability?

F. R. McMillan (*Chicago*)*:* We have all been thinking on that for the last two years. L. W. Walter is the chairman of an Institute committee* charged with the responsibility of reporting back as soon as we are able to report the conclusions which Mr. Hagy's questions have asked for; "What is the factor that determines the durability of concrete, the amount of cement per cubic yard or the water ratio or the

*Committee 114, Relations of Cement Content, Strength, Water Ratio and Durability.

strength in pounds per square inch? We have not the data that warrants us in making a report at this time but we have some information of interest. In our laboratory Professor Gonnerman is now running tests in which he uses four different cements. First he determines the age-strength relation for several different water ratios; that is, he finds out how one cement increases in strength with time when used at a certain water ratio. The same way for the other cements and the different water ratios. He is then taking some additional specimens, on the basis of his data, which represent a given strength of concrete, whether that strength comes from a low water ratio and early curing, to get high early strength, or whether it comes from a higher water ratio and longer period of curing. These specimens of a given strength are subjected to freezing and thawing cycles. When these data are available, we will have some basis for comparing the resistance of concrete of say 1700 p. s. i.—whether it had that strength on the basis of rapid hardening cement at early age or slower hardening cement at a greater age. He has used 1700, 3,000 and I think about 5,000 p. s. i. When we started this series, we thought we were going to get data rapidly, but complications entered. For example: take a high early strength cement giving 1,700 lbs. at 24 hours. That first freezing does not harm the concrete very much and the next day, while it is thawing out, it is getting some additional curing, and for the first two or three weeks that specimen is rapidly gaining strength. Later we have another specimen of the same strength which is probably sixty days old, of a slow hardening cement or of a leaner mixture. It goes into the freezer at 1,700 lbs. That cement does not respond to this extra curing the same as the high early strength cement does, because it is in the first few days that the response to the additional curing is most rapid. So, while we think we are comparing two specimens of the 1,700 lb. cement, we are comparing a 4,500 lb. specimen with one of 1,700 lbs. The result is, we are still not able to make a report to this group saying positively what the limitations are. I think that as Mr. Powers has said, the only ground we now have to go on, is that the character of the paste determines the water tightness on which we must rely; that is, the character is determined by the proportion of cement and water in the paste and the extent to which the hydration has progressed; in other words, you cure cement to get the greatest potential strength and use the water ratios, five or six gallon, depending on the exposure.

J. C. Pearson (Allentown, Pa.): I want to ask Mr. McMillan a question: is the paste of a high early strength cement of a given water ratio of a higher quality than one of ordinary cement?

Mr. McMillan: That is a question we are starting out to investigate. For the same degree of hydration, I think they are the same quality; in other words, for the same porosity, the same amount of uncombined water in a cubic inch of paste, I think they are of the same quality.

A Member: Is that from the standpoint of durability?

Mr. McMillan: Yes.

W. S. Thomson (Evanston, Ill.): Is the effect of vibration to be regarded seriously as disturbing the reinforcement? Professor Davis, I think, mentioned the possible floating of the reinforcement.

M. Lindau: My observation is that where the steel has been reasonably well secured, vibration has in no way disturbed it. To be sure that mortar completely surrounds reinforcing steel, where there is a great mass of steel, we have frequently put the vibrator on the reinforcing steel. This extends the vibrations the length of the bar, but it does not disturb its position unless the vibrator is dropped on it.

Herbert M. Shilstone (New Orleans): I was interested in problems of reinforcing in a building we were constructing in New Orleans. There was so much reinforcing in the bottom of the pans that the mortar could not penetrate to the metal screen that was suspended under the pan to form the beam between pans. When the forms were stripped, the beams were defective, reinforcing exposed, and the architect very much worried. The contractor had never used a vibrator, and I persuaded the architect to allow me to put a vibrator on the job. The steel was so close to the outer surface that you could not even get your fingers between the three-quarter inch metal and the forms. The result (shown by Mr. Shilstone from a stereopticon slide) is a clear demonstration of what a vibrator does in such circumstances. I followed the steel as the concrete was put in place. I wanted to be sure there was not the slightest movement of the steel out of position. We had been using eight men spading concrete, with a six-or seven-in. slump; and with the vibrator two workmen who never saw a vibrator before placed 2-in. slump concrete with perfect results using a Jackson spud-vibrator.

L. W. Walter (Jersey City): In the construction of concrete ships in 1918, continuous concreting operations were carried on for eight days; because of extreme difficulty in placing and the poor accessibility, placing was slow. This work was done at a season when the surrounding temperature was high; concrete was extremely rich in cement, setting quickly. We used vibrators on the forms and on the steel, so

that I am sure in many instances the concrete had taken its initial set before the next successive layer was placed. We used the Jack-hammer type of vibrator, applied with force to the reinforcing steel without any detrimental effect so far as we have observed then or since. The concrete was sufficiently compacted to preserve the steel from corrosion.

I think a little more might be said of the necessity for proper compacting of reinforced concrete, particularly in reinforced marine structures surrounded by salt-laden air and spray. Basically we look upon the water-cement ratio as the basic principle, but in marine structures, reinforced, many very serious mistakes have been made by failure to compact the concrete sufficiently below the steel, the trouble occurring usually in the lower steel of the beams and floor slabs, in the beams particularly. I would not want to undertake to build a reinforced concrete marine structure without the use of vibrators, and in the slab construction work I would depend chiefly on vibrating the steel and the floor supports, that is the false-work under the floor by the use of what would ordinarily be termed the exterior type of vibrator rather than interior type, particularly in thin slab construction.

In reference to discussion of this and other contributions to the subject of "Vibration as a Means of Placing Concrete" see Editor's note, page 48.

Discussion of Report of Committee 407:

Painting on Concrete Surfaces*

W. B. Roberts† (*Pittsburgh*—by letter): Dr. Anderegg's report is interesting and instructive. Of particular importance is his showing that paint failures may be caused by the disruptive action of crystallizing efflorescent salts at or on the surface. It would appear, however, that the use of a porous, inorganic type of coating which permits the concrete to "breathe" really encourages the rate of evaporation of the water solvent of these salts. True, these crystal formations may not all occur at a point where they cause lifting of the film but if they do deposit on the surface of the coating, they effectively destroy the appearance and decorative effect which was the principal reason for applying the paint. This is particularly true if these salt solutions carry a quantity of iron compounds, which may occur after the lime in the concrete has all been carbonated or leached away.

Dr. Anderegg states that these inorganic films are "not impermeable to moisture." If this is a fact, then they can afford but little protection to concrete surfaces that may be subjected to alternate freezing and thawing conditions. The effect of "frost" is probably one of the major causes of surface disintegration of concrete. If protection against this type of trouble is required, it would seem more logical to select for a coating material or coating system one which falls within the classification of the semi-permeable, film forming, organic compounds, such as drying oils, varnishes and tars.

It is possible by careful selection of the proper ingredients to produce paint films that are highly resistant to moisture passage. Two or three coats of such paints are almost impermeable to water vapor. Crystal growth behind such films therefore must necessarily be very slow if it occurs at all. The possibilities are that such a paint would offer many years' service before failure would result from these causes.

Dr. Anderegg in discussing vehicles suitable for concrete application infers that linseed oil is not very desirable for this purpose. More emphasis could have been placed on this point. Experience tends to

*Journal Amer. Concrete Inst., Sept., 1932: *Proceedings*, Vol. 29, p. 1.
†Aluminum Co. of America.

prove that linseed oil, as such, should never be used as the chief ingredient for paints of this character. It is so readily saponifiable that it is not uncommon to see such paint films on concrete actually dissolve or powder under the influence of condensed moisture and drop off shortly after application. Tung oil, particularly when heat treated and reinforced with varnish gums, does not exhibit this tendency, if its acid value is maintained at a low minimum. Paints made with tung oil varnishes containing phenol formaldehyde or similar synthetic resins are especially resistant to alkaline attack in addition to their other valuable characteristics of high impermeability, good adherence and duarability.

In connection with the bituminous-aluminum powder paint mentioned, a method of protecting and decorating concrete surfaces that must resist the action of weather is being investigated. From a theoretical standpoint it has considerable promise. Essentially, it consists of first coating or priming the concrete with a bituminous paint or gas tar well diluted with volatile solvents and following this with an application of a varnish or bituminous paint containing as a pigment aluminum bronze powder, similar to that described in the report. The purpose of the priming coat is to impregnate the concrete and seal the tiny interstices on and immediately below the surface of the concrete, leaving a negligible film on the face. After losing its volatile components it should effectively stop the moisture flow and retard "breathing." The top coats applied over the black primer are intended to serve the three-fold purpose of protecting the undercoat from disintegration by sunlight, reinforces its moisture-proofing characteristics and may be considered decorative in character. The aluminum pigment has the property of preventing the "bleeding" of the bituminous material and thus other different colored top coats may be applied if desired.

CONVENTION DISCUSSION BY CHAIRMAN COMMITTEE 407

F. O. Anderegg (Pittsburgh): In painting concrete surfaces, whether for decoration or for the so-called protection of the concrete, there are certain fundamental limitations that need to be considered and understood carefully before attempting to apply such coatings. The coatings that have been used fall into two general classes, those essentially inorganic in character and those essentially organic. Among the first are included cold water paints, portland cement paints and inorganic stains. Some examples of the two latter have been seen in the lobby.*

*In an exhibit of concrete surfaces and color possibilities sponsored by Committee 409, Recommended Practice in Architectural Monolithic Concrete Construction and set up by the Secretary of the Committee, W. E. Hart.

Among organic paints are those with a linseed oil base and those with a china wood oil base, those which contain some of the newly developed synthetic resins which have very interesting properties indeed, and certain bitumen paints which have been proposed for the protection of the concrete. Now then, the question is whether these organic paints are compatible with concrete in their nature. Concrete is essentially inorganic, and its character essentially water loving. These organic paints are essentially water hating. You take a concrete slab that has been freshly prepared and there is naturally present an alkaline condition capable of attacking a good many of the oils used in a good many organic paints, decomposing the oils and producing a disintegration of that material. When you are dealing with resins, you encounter the problem of getting them firmly bonded to the concrete. It is essential to secure mechanical anchorage into the surface and pores of the concrete. Even when you do that, comes the question, are you seriously interfering with the breathing of the concrete? If the concrete is so placed that moisture does not get into it, this question of breathing may not be so essential, but should the concrete be in contact with the soil or with water or in such a position that the rain water will get into it from behind, then you have a moisture condition there that is likely to raise serious complications. Unless your film is absolutely impermeable to moisture and unless it has been most firmly anchored to the concrete, you have the whole film in a great many cases pushed off bodily by the action which is expressively called "preferential wetting," that is, the concrete prefers to be wetted by water than by the organic paint, and the result is a complete pushing off of that material. Suppose, on the other hand, your film is partially permeable to moisture in spots, pinholes, etc.; the moisture will pass through. The moisture in concrete, especially in new concrete or very old concrete that has had a chance to accumulate a good many sulphates from the rain water by contact with the soils, contains those soluble salts, and when evaporation takes place, these soluble salts are deposited behind the pores, the crystals grow, continue to grow, until they become strong enough and numerous enough to push off the surface and you get a pitting there that can continue extensively all over the wall. You see that on the inside of a leaky brick wall where the water has come through the plaster and pushed off the paint that has been applied to the plaster or the wall paper. Now comes the question raised by two people who are interested in the development of protective coatings for concrete, as to whether the underlying principles I have been presenting here are correct or not. In the first place, Mr. Roberts, of the Aluminum company selling his aluminum leaf to paint manufacturers for the

manufacture of a paint for concrete and other materials, advances this argument; suppose you have, as we have in Pittsburgh, a concrete retaining wall or concrete lining of a vehicular tunnel through which a great deal of moisture has passed through the years of its life, and as it has come through, it has produced various disintegrating tendencies. The surface of those walls looks bad. Mr. Roberts and some other people in Pittsburgh interested in bituminous coatings, have proposed to apply to such concrete and also to bridge abutments, etc., a coating of a bituminous character which penetrates well into the concrete, to secure a good anchorage for other coats of bituminous matter, and finally a coat of bitumen in which the aluminum has been suspended and which leaks out and gives this characteristic surface of aluminum paint and oil; the idea behind this being that you will make the wall look apparently all right and make it seem to give good service for a number of years. My objection to that argument is that you bottle up the pores and reduce the breathing and hold within the wall a continuous supply of moisture which is a disintegrating agent bringing about the disintegration of the wall, and, as has been shown with a great variety of other building materials, such painting actually promotes the disintegration. A letter from a member of the Dupont organization, interested in another waterproof treatment, has advanced similar arguments. The objections to them that I have raised are, I think, of sufficient importance to cause some hesitation in the use of indiscriminate materials for application on portland cement concrete. Now a word or two about the inorganic treatment. The cold water paints usually contain a glue or casein which, on wet concrete, undergoes disintegration and soon falls off. A portland cement paint is a neat cement applied in a very thin layer, and when it can be cured adequately it gives very good results. The difficulty is in taking pains enough to get proper curing. It can be done and will give you very satisfactory results.

The stains penetrate within more or less porous concrete and undergo reaction with the lime and deposit there and produce several interesting possibilities.

Discussion of a paper by Mark Morris:

"Mortar Voids Method of Designing Concrete Mixtures"*

AUTHOR'S CLOSURE

Mark Morris (*Ames, Ia.*—by letter): Messrs. Jackson and Tremper both called attention† to the fact that with the use of some abnormal coarse aggregates it is impossible to predict the strength of the concrete from the strength of the mortar. The author is willing to admit that this may be the case in some circumstances. He is of the opinion, however, that the mortar voids method of design will permit a more accurate valuation of the effect of abnormal coarse aggregates than any other method now available.

Since the characteristics of the mortar can be definitely established the effects of the coarse aggregate may be more readily evaluated. This can then be done without the interference of the effects of other variables, such as the water-cement ratio.

It would appear to us, in Mr. Tremper's table, furnished with his discussion, that perhaps the low strength of concrete obtained with crushed rock, A, was due in part to the high water-cement ratio, and in part to the nature of the aggregate. Mr. Tremper neglected to state the number of specimens used in his test, but we assume that there were a representative number.

We also call attention to the fact that the compressive strength and the flexural strength give different relative values for some of the coarse aggregates. Some recent work in our laboratories has convinced us that the differences in the relative values of compressive strength and flexural strength may be traced to differences in the moisture condition of the specimens at the time of test. In all of the work upon which the paper was based, particular attention was given to the moisture condition of the specimens at time of test. This was found to be absolutely necessary for checking results, even with the same material. We are calling this to Mr. Tremper's attention for his assistance in interpreting our results, and his own.

*Journal, Amer. Concrete Inst. Sept. 1932, *Proceedings*, Vol. 29, p. 9.
†Journal, Amer. Concrete Inst., January 1933; *Proceedings*, Vol. 29, p. 241.

Mr. Jackson has discussed extensively his experiences with a particular aggregate, which, from its behavior in the two strength tests, is undoubtedly abnormal. We fail to see where the experiences of either of these two men have done other than to emphasize the value of the method of designing concrete mixtures.

Discussion of Report of Committee 105:

"Reinforced Concrete Column Investigation"

*Tentative Final Report of Committee 105**

and

*Minority Recommendation for Design Formula of Reinforced Concrete Columns**

R. T. Logeman (*Chicago*—by letter): Upon reviewing the "Tentative Final Report of Committee 105" on the "Reinforced Concrete Column Investigation," my first thought is disappointment that the Committee did not withhold its report, and continue its studies until they could present a united recommendation. To have the Committee split on the basis of five members submitting one report, and two members submitting a dissenting report, only implies that the investigation was not 100 per cent conclusive. This, however, in no way detracts from the credit due the entire Committee for its painstaking labors spread over a number of years. On the contrary, it rather focuses attention on the long recognized fact that an analysis of reinforced concrete columns is not a simple thesis. With the Committee divided, it presages a divided opinion among the Institute members, and a much more serious division among engineers and architects at large who look to the Institute for correct guidance in concrete construction. Engineers carry the responsibility for safe construction and should not be tempted with the unknown nor experiment with sound construction principles. Engineers should know the weak link in their structures and not temporize but remove it.

The main differences between the majority and minority reports appear to be confined to two features: one pertaining to the function of the spiral as represented in the third and last term of its proposed column formula, and the other to the "Factor of Safety."

In regard to the action of the spiral, the experiments are far from being conclusive. It may be gratifying to know that the spiral adds something to the load carrying capacity of the column, but the amount added is largely confined to the "Region of plastic deformation:" the region where stress and strain cease to be proportional, and where, consequently, the amount of added load carrying capacity is too un-

*Journal, Amer. Concrete Inst., Feb. 1933; *Proceedings*, Vol. 29, p. 275-284.

certain. When one considers that the amount of the carrying capacity or influence of the spiral was determined by a process of elimination, that is to say, by subtracting from the ultimate column strength the assumed capacity of the vertical steel at its yield point strength and the concrete at 85 per cent of its control cylinder strength, this brings to light the action of the spiral in the range of failure loadings. It is enlightening to learn that the spiral comes into action at the extreme loading stages, but the designer of columns wants to know something about its action at ordinary working loadings. In other words, he wants to know the effect of the spiral on the supposed combined elastic action of the vertical steel and concrete at working loads. If the action of the spiral steel is not definitely known at working loads, is it rational to assume that it is in some manner to be proportioned from its action at failure loads? At failure the experiments indicate that 1 per cent of spiral was as effective as approximately 1.5 per cent to 2.5 per cent of vertical reinforcement. This is rather a wide spread, but, based on these findings, the Committee selects a mean of 2.0 as a constant "k" in the proposed formula for determining the ultimate load attributed to the spiral, while, as a matter of fact, this load is not at all in the nature of a vertical load carried by the spiral, but, in reality, is considered as an increment load to be added to the concrete or in some combined action, to the concrete and vertical steel. Regardless of how the spiral may transmit its contribution to the concrete or steel, or in some manner to both, and perhaps, eventually, due to plastic yielding, to the vertical steel, the important question for consideration is, if the spiral is of no value within the limits of ordinary loading, what practical value is there in introducing the failure value of the spiral into a column formula when a finished structure is designed not for failure, but for working loads?

The present American Concrete Institute Building Code of 1928 does not recognize the spiral as forming any part of the design column formula; and there is nothing conclusive, based on the pending investigation or otherwise, justifying the Committee in proposing such an addition in its new type of column formula.

The increase in the steel stress and the decrease in the concrete stress, due to plastic yielding, and the fact that this redistribution of stress has no effect upon the ultimate strength of a column when tested to failure, can be viewed as very interesting phenomena, but again, the designer is bound to pursue his quest for more light. It has not been conclusively shown that the vertical steel does not carry load in excess of its yield point strength; as a matter of fact, the tests indicate that

the deformation of the vertical steel ranges anywhere from 2.5 to 10 times the deformation at yield point, which can mean but one thing; namely, an increase in the burden of the vertical steel. If, in the final analysis, the vertical reinforcement takes on an over-burdened load—carrying duty, the question naturally arises as to what is the real behavior and duty of the concrete and spiral. The experiments seem to show that at low loads, the vertical steel and concrete deform together, with the spiral steel maintaining, so to say, the integrity of the concrete, but otherwise dormant or not in action. At higher loads, the deformation, between vertical steel and concrete at the ends of the column, seemed to continue to increase equally, but near the center of the column the deformation in the steel exceeded the deformation in the concrete, and continued to increase until failure occurred, the spiral at that stage coming into action. This may indicate that as the three elements come into action, (vertical steel, concrete and spiral) the vertical steel is no longer held in perfect alignment by the concrete and spiral. The first failure of the Quebec bridge was due to just such a condition—a lack of lateral strength to preserve the alignment of the main material. Similarly, the careful designer wants to have further assurance of the proper interaction of the three materials comprising the column make-up. After the elastic properties of the concrete are destroyed, it may, as the experiments indicate, still be able to carry vertical load, due primarily to the encasement action of the spiral, but neither the concrete nor the spiral can efficiently carry the horizontal stress resulting from the bending of the vertical steel, to the end reaction points of the column. The concrete may so function to the limits of plain concrete in tension and shear, provided these properties are not destroyed due to over loading, but the spiral has no such properties at all. The case is similar to a steel column where the main carrying members are inefficiently tied together with horizontal batten bars, instead of diagonal lacing or a continuous plate.

The use of a higher yield point vertical steel, as has been suggested by the Committee and others, looks, at first thought, as a natural outcome of the tests, but such a step would tend to further complications because it presupposes that the additional force necessary to hold the vertical steel in alignment under the additional vertical load can be taken care of by the concrete and spiral, keeping in mind, also, in this connection, that the higher yield point steel is supposed to lead to a smaller diameter column and consequently to a column with a lower lateral resistance. According to the proposed formula, the strength of the column, for any given grade and strength of concrete,

is established without giving any consideration to the effect upon the concrete of using different yield point grades of vertical steel. Bearing in mind that the "Modulus of Elasticity" of all grades of steel is practically a constant, (assumed at 30,000,000 p. s. i.) which is to say that the deformation of the vertical steel is in direct ratio with its unit stress, the proponents for a higher yield point steel cannot overlook the effect that a difference of deformation of the vertical steel has upon the deformation and strength of a given grade of concrete and spiral. Certainly no designer can ignore the effect of deformations upon the respective stresses produced in the steel and concrete, even though the experiments and the "plastic flow phenomena" are, apparently, knocking into a cocked hat the time honored stress or "n" ratio—a ratio based on equal deformations and constant moduli of elasticity of steel and concrete.

By no stretch of the imagination can the most ardent advocates of the "plastic flow" theory attribute to the concrete an ability to adjust itself to stress conditions in the steel entirely independent of such effect upon the stress conditions in the concrete itself, and yet that is exactly what the proposed formula permits. As an extreme case, after determining the stress in the concrete based upon any assumed f_c' value for the concrete, and percentage (p) for the steel, suppose that one-half of the vertical steel has a yield point stress (f_y) of 40,000 p. s. i. and the remaining one-half a yield point stress of 80,000 p. s. i.; an extreme case to be sure, but focusing attention upon the question of how the plastic flow of the concrete can adjust itself to the two different stress and deformation conditions of the vertical steel.

The mass of data that has been gathered by these tests should be given further study; if necessary, the experiments should be continued until more definite conclusions can be established as to the inter-relation of the vertical steel, concrete, and spiral. Until this is done the Institute should not make a final recommendation either to its members or to the Engineering profession at large.

Walter H. Wheeler (*Minneapolis*—by letter): The recommendations for reinforced concrete column design proposed by this Committee are revolutionary in character and if adopted and incorporated into building codes will permit the use of reinforced concrete columns considerably smaller in diameter and carrying much heavier loads per square inch of cross sectional area than any existing code.

One of the important differences between the proposed code and existing codes is the inclusion of the outer shell of a spirally reinforced column in the effective area of the column. Will the fire underwriters approve such a design for fire-proof buildings?

TABULATION A—SHOWING SAFE LOADS ON TWO SIZES OF COLUMNS BY PROPOSED CODE, PRESENT A. C. I. CODE, WHEELER METHOD

Column Size	Core Diameter	Concrete Ultimate Strength	Vertical Bars	Spiral	Safe Load 16.	Code
24x24	21″ rd.	3000	8—1″ sq.	⅜″—21″ d.—1½″	613,000	Proposed Code
24x24	21″ rd.	3000	8—1″ sq.	¼″—21″ d.—1½″	364,000	Present A. C. I.
24x24	21″ rd.	3000	14—1⅛″ sq.	½″—21″ d.—2⅞″	613,000	Present A. C. I.
24x24	21″ rd.	3000	8—1″ sq.	⅜″—21″ d.—1¾″	611,000	Wheeler
16x16	13″ rd.	2000	4—1″ sq.	⅜″—13″ d.—2½″	218,000	Proposed A. C. I.
16x16	13″ rd.	2000	9—¾″ rd.	¼″—13″—2″	142,000	Present A. C. I.
16x16	13″ rd.	2000	9—1″ rd.	⅜″—13″—2½″	218,000	Present A. C. I.
16x16	13″ rd.	2000	4—1″ sq.	⅜″—13—2½″	205,000	Wheeler

TABULATION B—SHOWING MAXIMUM LOADS ON TWO SIZES OF COLUMNS BY PROPOSED CODE, A. C. I. CODE, WHEELER METHOD

Column Size	Core Diameter	Concrete Ultimate Strength	Vertical Bars	Spiral	Safe Load	Code
24x24	21″ rd.	3000	20—1¾″ rd.	⅜″—21″—1½″	1,470,000	Proposed Code
24x24	21″ rd.	3000	16—1⅛″ sq.	½″—21″—2½″	692,000	Present A. C. I.
24x24	21″ rd.	3000	15—1″ sq.	$\frac{7}{16}$″—21″—2″	686,000	Wheeler
16x16	13″ rd.	2000	13—1¼″ sq.	⅜″—13″ —2½″	650,000	Proposed Code
16x16	13″ rd.	2000	8—1″ sq.	⅜″—13″ —2¼″	240,000	Present A. C. I.
16x16	13″ rd.	2000	9⅞″ rd.	$\frac{5}{16}$″—13″—1¾″	229,000	Wheeler

For comparison, I have prepared two tabulations, A and B. A 24-in. column and a 16-in. column is used in each tabulation. It will be noted (see Tabulation A) that a 24-in. column of 3000-lb. concrete with eight 1-in. square vertical bars and the required spiral will have a safe load capacity of 613,000 lbs. according to the proposed code and 364,000 lbs. according to the present A. C. I. Code and a safe load capacity of 611,000 lbs. if designed according to the method which the writer has used for many years in localities which have no code. To bring the safe load capacity of this column up to 613,000 lbs. under the

present A. C. I. Code would require 14 1⅛-in. square verticals and ½-in. spiral 21 in. diameter at 2⅞ in. spacing.

It will be noted (see Tabulation A) that a 16 x 16 column of 2,000 lb. concrete with four 1-in. square verticals and the required spiral will have a safe load capacity of 218,000 lbs. under the proposed code, and a safe load capacity of 205,000 lbs. designed according to the method used by the writer. With nine ¾-in. round verticals and the required spiral this column would have a load capacity of 142,000 lbs. under the present A. C. I. Code. It would require nine 1-in. round verticals in this column to bring the safe load capacity up to 218,000 lbs. under the present A. C. I. Code.

Tabulation B shows the maximum safe load capacities for 24-in. columns of 3,000 lb. concrete and 16-in. columns of 2,000 lb. concrete designed in accordance with the proposed code, the present A. C. I. Code and the method used by the writer, which is the same as the existing Minneapolis Code for 2,000-lb. concrete, but employs higher values for higher strength concrete.

It will be noted that a 24-in. column of 3,000 lb. concrete can be made to carry a maximum safe load of 650,000 lbs. under the proposed code, 240,000 lbs. under the present A. C. I. Code and 229,000 lbs. by the writer's method.

The proposed code will materially reduce the cost of reinforced concrete columns and to that extent it certainly is desirable. With the lower percentages of vertical steel, it will, in the writer's opinion, result in safe and economical designs. With the maximum percentages of vertical steel the safe load capacities of concrete columns of given diameters will be increased so much in excess of anything that has been common practice that it may be questioned whether such a radical advance in one step is desirable without experience other than laboratory tests.

L. J. Mensch (*Chicago*—by letter): This important series of tests ought to be considered proof that the classical laws of elasticity do not apply to reinforced concrete construction. The ratio n and all involved formulas ought to be discarded. Evidently the stresses measured by extensometers in short time tests lead to but inaccurate pictures of the underlying mechanics. This applies to girders and slabs just as well as to columns. As soon as an attempt is made to formulate the behaviour of columns or girders under long continued loading, or near the yield point in a fast loading test, based on stresses measured at the working load under fast loading, our endeavors break down and we are compelled to resort to empirical formulas, which, if properly devised, are

safe guides. There does not exist today an exact theory of the strength of any structure.

Formula 1 was presented to the Institute as early as 1915 by A. N. Talbot in a slightly different form. It was then a bold departure from existing rules; unfortunately it never was adopted by any building code. In the present form this formula was first published by F. Emperger (Heft 11, Mitteilungen ueber Versuche, *Oesterreichischer Eisen Beton Ausschuss*, 1927) and clearly establishes the influence of the yield point of both vertical and spiral reinforcement on the strength of columns. Until very recently nearly all experts of reinforced concrete denied the markedly favorable influence of steel of a high yield point on the strength of reinforced concrete structures, and actually, there does not exist a single code in the U. S. which gives proper credit to steel of a higher yield point than 60,000 p. s. i. The writer has used hundreds of tons of steel of a yield point higher than 120,000 p. s. i. and has experimented with steel of a yield point higher than 200,000 p. s. i. More than 30 years ago Professor Considere first proved the favorable action of hard steel in reinforced concrete construction by a long series of tests. It does not seem rational to consider as yield point of a column the load given by formula 2. More careful observations of many tests will show that for percentages of vertical steel as commonly used the quality of the concrete governs the yield point, and this varies widely although the ultimate strength might not show large differences. Very interesting and important is the conclusion from these tests that the ultimate carrying capacity of a column when tested in a testing machine for a few hours is only 5 per cent lower than in fast loading, and that we may expect that long continued loading will not decrease the ultimate load more than 10 to 15 per cent.

Formula 3 will lead to rather strange column design, as will be shown by an example. The writer often used 20-in. square columns of 1:1:2 concrete in tall buildings. Assuming $f_c' = 4000$, $f_s' = 40{,}000$, $A_g = 400$, $A_c = 201$ it follows from formula 3, $p' = 4.3$ per cent, or a ½-in. spiral with 1.15-in. pitch. This close spacing does not please the committee, even a ⅝-in. spiral with 1.78-in. pitch would not come within its requirement, therefore, we would be compelled to use a ¾-in. spiral with 2.55-in. pitch. Such spirals are hard to obtain and high in price.

The great simplicity of formula 7 may count as a merit; however, it is not as plausible as formula 1, and, besides, often entails as above shown, very high percentages of spiral reinforcement and a rather low factor of safety. Most columns are eccentrically loaded. Formerly,

when higher factors of safety were used, the tedious calculations, involving the solution of a third degree equation according to established rules, was properly not gone into and the eccentricity entirely neglected; the fact that long time loading might reduce the factor of safety was also properly neglected. With a proposed factor of safety of less than 3 we may expect serious trouble with columns when formula 7 gets into the hands of the average designer and the work is done by the average concrete constructor.

The proposed working formula is evidently based on the tests of 14 columns of series 6. An inspection of fig. 26 and 21, *Proceedings*, Amer. Concrete Inst., Vol. 28, p. 312 and 314, will show that columns with protective shells reach ultimate loads at an unit deformation from .001 to .0015, while naked spiraled columns fail at an unit deformation of about .0025; we further find that the stresses in the spiraled core at the unit deformation at which the practical column fails is about $^9/_{10}$ of the maximum stress it can carry without the protective shell, and other tests show that the protective shell begins to crumble away at loads which are 25 per cent and less of the naked column. We can, therefore, say that the hooped concrete column with protective shell may have a large factor of safety against complete breakdown and might have only ½ of that factor of safety against breaking away of the protective shell, or at least against serious cracks. The committee does not explain the reason for this cracking, or that concrete columns with longitudinal reinforcement only are less reliable than spiral columns. The latter class of columns fails always as soon as the yield point of the steel is reached for the reason, hardly known in this country, that Poisson's ratio becomes greater than 1 as soon as the yield point is reached and splits off the concrete. Ties placed 6 in. or 12 in. apart can hardly prevent such cracking. Some tests have been made with only one longitudinal bar placed in the center; they invariably gave higher test results than columns with the bars placed in the corners. It is also clear that a closely spaced spiral will prevent such cracking when the bars are stressed beyond the yield point. But while the core of the practical column does not split up as soon as the yield point of the longitudinal bars is reached, the protective shell splits for the reason that the lateral deformation of the shell at a stress near the ultimate of plain concrete is ⅓ or ½ of the longitudinal deformation, while the concrete of the hooped core might have a Poisson's ratio of less than 1/5 at the same concrete stress (McKibben and Merrill, *Proceedings*, Amer. Concrete Inst., Vol. 12, p. 220), which means that there will be a tendency for the protective shell to separate from the

core, and besides, place this shell under circumferential tension. A careful analysis of the columns of series 6 shows that the protective shells failed in some cases at less than sixty per cent of the stress of the corresponding ultimate stresses of the test cylinders; further considering that in the practical column the outside shell is often of inferior concrete, it is a rational assumption to ascribe to the concrete of the shell a stress at the ultimate load which is only one half of the ultimate stress of a test cylinder. For the considerations just mentioned, and in view of the tests of series 6, the writer proposes as a rational formula for the strength of a hooped concrete column

$$P = \tfrac{1}{2}(A_g - A_c) f_c' + .85\, A_c f_c' + \frac{9}{10} A_c p f_y + \frac{9}{10} \times 2 \times p' f_s' A_c$$

He would allow a factor of safety of 4 on that part of the column depending on uncertain workmanship, namely the concrete and a factor of safety of 3 on that part depending on the steel.

The permissible load on the column would be

$$P = \tfrac{1}{4} \times \tfrac{1}{2} \times (A_g - A_c) + \tfrac{1}{4} \times .85\, A \times f_c' + \frac{3}{10}\, p \times f_y\, A_c + \frac{6}{10}\, p'\, f_s' \times A_c$$

or the unit stress on the core part of the column

$$\frac{P}{A_c} = \left(\frac{R}{8} + .0875\right) f_c' + \frac{3}{10} p \times f_y + \frac{6}{10}\, p' f_s'$$

but in no case ought the sum of

$$\tfrac{1}{4} \times \tfrac{1}{2} \times (A_g - A_c) f_c' + \tfrac{1}{4} \times .85\, A_c \times f_c' + \frac{6}{10}\, p' \times f_{s'} \times A_c$$

be greater than

$$A_g \times \frac{f_c'}{2.5}$$

The minority report rejected with justice formula 7 but offered in its stead a formula leading to just as impracticable spirals as the majority report. Both reports limit the vertical steel to 8 per cent. This is evidently a guess and not based on this investigation, nor on properly made tests by other investigators, which have shown that even 40 per cent and more longitudinal reinforcing behaves properly in spiralled columns. For the same reason the guess of the minority report to limit structural steel in hooped concrete columns to 20 per cent must be emphatically rejected. The tests mentioned in the writer's paper on composite columns were not properly studied by the minority of the

committee. The committee did not make tests on spirals with a very high yield point, but enough tests have been made both here and abroad on wires where f_s' equals 150,000 p. s. i. Such wire is a common article of commerce here and costs hardly $10 a ton more than hot rolled wire; in times like this we have to take advantage of every point which saves cost, but the committee did not encourage its use. The report says that formula 1 is valid for longitudinal steel with a yield point of 96,000 p. s. i. The practical column cannot deform enough to utilize a deformation of .0032 before the protective shell cracks; its use is of advantage only in increasing the factor of safety against complete breakdown, unless as might be done in ribs of very long bridges the protective shell is applied by means of a cement gun after all dead load is on, or unless the protective shell consist of a more pliable material. In this case the shell ought to have its own spiral reinforcement of closely spaced thin wires.

J. Di Stasio (*New York*—by letter): In the Tentative Final Report of Committee 105, Reinforced Concrete Column Investigation, the majority and minority recommendations differ as to the design formulas proposed. Fundamentally, both are derived directly from the results of tests as expressed, first, by basic equation 1, which gives the ultimate strength of any reinforced concrete column, and, second, by basic equation 2, giving the yield point of all columns and the ultimate strength of tied columns. The majority report design formula 7 allows 25 per cent increased stress in a spiral column over a tied column as given in formula 9, but provides no credit for spiral percentages greater than a minimum—equation 8. Minority report formula 3 recognizes increase in ultimate strength with increase of spiral, but limits the increase in working load due to spiral to 40 per cent of working load without spiral. The majority report formula 4 for spiral columns is simply a special case of the minority report formula in which spiral strength is taken 15 per cent greater than the strength of the shell, and becomes formula 7 when modified by a factor of safety varying with time yield. If the basic equations 1 and 2 are accepted as the fundamental premises expressing the results of tests, the majority report formula does not reflect a complete interpretation of these tests as it limits a spiral column to the special case where spiral strength is equivalent to 115 per cent of the shell strength. While the elimination of the spiral term in the design formula recommended by the majority report results in an equation convenient to combine with bending, this would be equally true for the equivalent special case of the minority report. After conducting such a broad series of experiments and developing therefrom equations 1 and 2, it seems illogical to limit design

to a special case, and it is believed that working formulas should include the entire range of the tests as expressed in the minority report.

Results of experience with columns in use, and the fact that ultimately the vertical steel carries its share of load irrespective of time yield, do not warrant a variable factor of safety dependant on per cent of vertical steel. In view of the greater toughness of spiral columns as disclosed by tests, it seems logical to reduce the yield point factor of safety with increase of spiral as indicated by formula 4 of the minority report. The effect of the different viewpoints regarding factor of safety is evident from a few examples. Assuming f_c = 3000 p. s. i., f_y = 40,000 p. s. i., f_s' = 60,000 p. s. i., R = 1.33, the following comparison results:

	Majority Report	Minority Report
	Spiral Columns	
%	$\frac{P}{A_g} = .25\,f_c' + .45\,f_y\,P_g$	$\frac{P}{A_g} = \frac{1}{3.5}\left[.85f_c'\,(1\text{-}P_g) + f_yP_g + \frac{2f_s'}{R}\left(P_a - \frac{.85f_c'\,(R\text{-}1)}{2f_s'}\right)\right]$
$P_g = .01$, $P_a = \frac{.43\,f_c'\,(R\text{-}1)}{f_s'} = .0072$	750 + 180 = 930 p. s. i.	720 + 114 + 3 = 837 p. s. i.
$P_g = .01$, $P_a = .02$	750 + 180 = 930 p. s. i.	720 + 114 + 332 = 1166 p. s. i.
$P_g = .08$, $P_a = .0072$	750 + 1440 = 2190 p. s. i.	670 + 914 + 3 = 1587 p. s. i.
$P_g = .08$, $P_a = .02$	750 + 1440 = 2190 p. s. i.	670 + 914 + 332 = 1916 p. s. i.
	Tied Columns	
	$\frac{P}{A_g} = .2\,f_c' + .36\,f_yP_g$	$\frac{P}{A_g} = \frac{1}{3.5}\left[.85\,f_c'\,(1\text{-}P_g) + f_yP_g\right]$
$P_g = .005$	600 + 72 = 672 p. s. i.	726 + 57 = 783 p. s. i.
$P_g = .03$	600 + 480 = 1080 p. s. i.	707 + 343 = 1050 p. s. i.
$P_g = .04$	600 + 480* = 1080 p. s. i.	700 + 456 = 1156 p. s. i.

*Maximum.

Thus, there is a fairly wide difference between the two reports throughout the range of spiral columns, and for tied columns with light vertical steel. With particular reference to tied columns, the minority report assigns somewhat more than usual credit to the concrete, while in the majority report, the strength assigned to the larger percentages of vertical steel appears excessive if restrained only by widely spaced lateral ties. But, in general, it is believed that the values resulting from the minority formulas more correctly represent the safe strength of the columns considered.

For practical fireproofing and economical reasons, a 1½-in. or 2-in. shell is generally used. For relatively small diameter columns designed in accordance with the majority design formula, and especially for high

strength concrete with hot rolled spirals, considerably more than 2 per cent of spiral reinforcement will be required to develop shell strength according to formula 8, and may reach 4 to 5 per cent in extreme conditions. From tests, the minority report limits the maximum total spiral to 2 per cent as the factor k drops off rapidly with increase beyond this amount. In deriving formula 8, the factor k equal 2 was used, but as no limitation is placed on maximum spiral, the majority report does not truly represent the shell strength of relatively small diameter columns. It seems unfortunate that small spiral columns are uneconomical, but, nevertheless, the 2 per cent limitation of the minority report apparently reflects the results of tests to date. In this connection, the minority 2 per cent limitation of spirals is evidently based on cold drawn wire. If dependant upon elongation, it would seem that the limit should be changed for hot rolled bars. This point is covered in the majority report where percentage varies with the useful limit of the spiral used.

Vertical steel is also a subject of difference between the two reports. For tied columns, the majority set the maximum per cent at 3 as opposed to 4 in the minority report and many current codes. Also, in both reports, if percentage of vertical steel is based on gross area, it is not clear why the minimum limit is 1 per cent for spiral columns as opposed to .5 per cent for tied columns.

Details are deserving of especial consideration. While the majority recommend that not less than 4 vertical bars be used, the writer suggests that size of bar to be also limited to ⅝ in. or larger to avoid kinks prevalent in smaller sizes. Column ties are not defined; some differentiation should be given depending on the number, size, and also the amount of vertical steel—foreign practice for example requiring an increase in size or closer spacing with more than about 10 sq. in. of verticals; desireable arrangements of cross ties should be indicated, and as insurance against poor workmanship and induced bending, it is good precaution to require one half normal spacing at the splices. For spiral columns, minimum shell thickness has not been specified.

With regard to nomenclature, majority report formula 2, apparently contains a typographical error in that $(1\text{-}p)$ should be $(1\text{-}p_g)$. To provide consistency between the two reports, it is suggested that the notation of the minority report be revised, changing p to p_g, p' to p_a, p'' to p_b, and p_a to p', making $p' = p_a + p_b$. Thus, p_g will clearly indicate per cent of gross area, and p' will correspond in the formulas of both reports.

In basing design formulas on ultimate strength of the concrete plus yield point strength of the steel, all divided by a factor of safety, the present accepted methods of designing for bending and direct stress no longer strictly apply. The design load is not a function of stresses actually occurring in the structure, and cannot consistently be combined directly with stresses resulting from actual bending moments. Also it is probable that spiral columns, when subjected to bending, will display quite different behavior above the yield point than when exposed to concentric load alone. It is therefore desireable to make exhaustive tests on the effects of bending before formulas are finally accepted as adequate for design.

For convention discussion by the authors of the "minority report" and closure by the Chairman of Committee 105, readers are referred to the JOURNAL *for November-December 1933.*

The Freyssinet Method of Arch Construction Applied to the Rogue River Bridge in Oregon

By Albin L. Cemeny and C. B. McCullough. Published October 1932; Discussion February, 1933. Author's closure is scheduled for publication November-December issue, 1933.

Stresses at a Crack, Size of the Crack, and the Bending of Reinforced Concrete*

BY H. M. WESTERGAARD†
MEMBER AMERICAN CONCRETE INSTITUTE

SYNOPSIS

A study of the individual crack leads to a revision of the conventional picture of the stresses in a reinforced concrete beam. At a crack the compressive strains in the direction of the beam are not proportional to the distance from a neutral axis. The revised picture suggests the relative advantage of the smaller sizes of reinforcing bars.‡

NOTATION

h, a, c, k, u, v = dimensions and distances shown in Fig. 3.
b = width of the rectangular cross section of the beam.
l = span of the beam.
A = total area of the cross section of the steel.
D = diameter of the steel bars
E, E_s = moduli of elasticity of the concrete and steel, respectively.
I = moment of inertia of the compound cross section, including the total area of concrete and $\frac{E_s}{E}$ times the area of the steel.
r, θ = polar coordinates, as shown in Fig. 2 and Fig. 3.
$\sigma_r, \sigma_t, \tau_{rt}, s, s_c$ = stresses, as shown in Fig. 2 and Fig. 3.
F = Airy's stress function.
$K, K_1, K_2, \ldots$ = constants.
C = total compressive force in the cross section at the crack.
T = total tension in the steel at the crack.
B = resultant of the bond stresses on one side of the crack.
M = bending moment.
z = width of the crack at the distance r from the vanishing point of the crack.
Z = width of the crack at the steel.
z_a = width of the crack at the bottom of the beam in Fig. 3.
α = angular opening of the crack at the bottom of the beam in Fig. 3.
w = elastic weight due to the crack.

*Received by the Secretary, Sept. 7, 1933. To be presented at the 30th Annual Convention, Toronto, Feb. 20-22, 1934.
†Professor of Theoretical and Applied Mechanics, University of Illinois, Urbana, Ill.
‡This study is an outgrowth of an earlier effort to analyze cracks in dams. This effort was made while the writer was on leave of absence from the University of Illinois, in the service of the Bureau of Reclamation, engaged in technical studies for Boulder Dam. Acknowledgment is made to Dr. Elwood Mead, Commissioner of Reclamation, R. F. Walter, Chief Engineer, and J. L. Savage, Chief Designing Engineer of the Bureau of Reclamation.

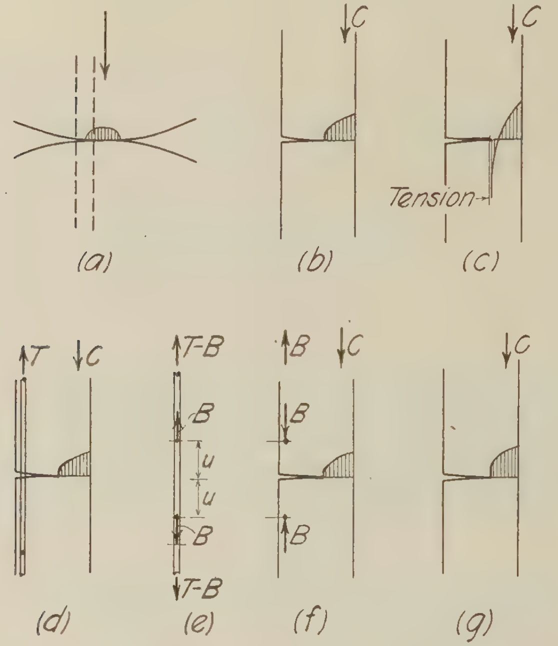

FIG. 1—STRESSES AT CRACK.
(a) Bearing pressures between cylinders. (b) Plain concrete wall with horizontal crack. (c) Same if the load were shifted without extension of the crack. (d) Reinforced wall. (e) Forces on steel. (f) Forces on concrete. (g) Stresses at ultimate strength

SUGGESTION FROM PRESSURES OF CONTACT BETWEEN CYLINDERS

Heinrich Hertz* showed in 1881 that the diagram of pressures of contact between two elastic cylinders is an ellipse, as drawn in Fig. 1(a). If the cylinders are imagined to be very large, the part between the vertical dotted lines may be interpreted as a vertical wall with a horizontal crack. This interpretation suggests the picture in Fig. 1(b): The eccentric load C causes a horizontal crack and a nearly parabolic diagram of compressive stresses in the section at the crack, the axis of the parabola being horizontal. A shifting of the load C toward the left would close a part of the crack. A shifting toward the right could be represented by the addition of a clockwise couple, which according to the theory of sharply curved beams would cause very great stresses at the end of the crack, as indicated in Fig. 1(c). In a concrete wall the inevitable effect is an extension of the crack until the type of diagram in Fig. 1(b) is restored.

*Heinrich Hertz, Crelles Journal für reine und angewandte Mathematik, Vol. 92, 1881, p. 156 (also in his Gesammelte Werke, Vol. 1, 1895, p. 155). A. E. H. Love, Mathematical Theory of Elasticity, third edition, 1920, pp. 191-196. A. and L. Föppl, Drang und Zwang, Vol. 2, second edition, 1928, p. 213.

When the wall is reinforced, as indicated in Fig. 1(d), a crack may develop of the same size as in Fig. 1(b), if the load on the wall is the resultant of the pressure C and an upward force T equal to the total tension in the reinforcement at the crack. The steel is pulled out at the crack. If u is the distance from the crack to the center of gravity of the diagram of bond stresses, the forces on the steel may be pictured as in Fig. 1(e), and the forces on the concrete as in Fig. 1(f). Close to the ultimate strength the diagram of stresses in the concrete may be modified as indicated in Fig. 1(g).

POSSIBLE STRESSES AND DEFORMATIONS AT A CRACK

A plane state of stresses is expressed conveniently by Airy's stress function. In terms of the stress function F and polar coordinates r, θ the normal stresses σ_r and σ_t and the shearing stress τ_{rt} in Fig. 2 are written as follows*:

$$\sigma_r = \frac{1}{r}\frac{\partial F}{\partial r} + \frac{1}{r^2}\frac{\partial^2 F}{\partial \theta^2}, \quad \sigma_t = \frac{\partial^2 F}{\partial r^2}, \quad \tau_{rt} = -\frac{\partial}{\partial r}\left(\frac{1}{r}\frac{\partial F}{\partial \theta}\right) \qquad (1)$$

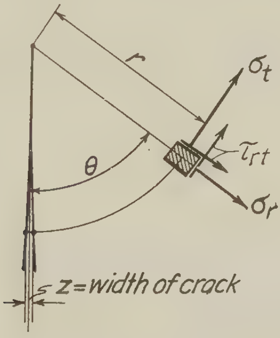

FIG. 2—STRESSES IN REGION OF CRACK

If the material can be considered as elastic, homogeneous, and isotropic, then F must satisfy the following equation at all points:

$$\left(\frac{\partial^2}{\partial r^2} + \frac{1}{r}\frac{\partial}{\partial r} + \frac{1}{r^2}\frac{\partial^2}{\partial \theta^2}\right)^2 F = 0 \qquad (2)$$

Assume a crack along the line $\theta = 0$. The following stress function is examined first:

$$F = \frac{K}{15} r^{2.5} (\sin 2.5\,\theta - 5 \sin 0.5\,\theta) \qquad (3)$$

in which K is a constant. Equations (1) give

*See, for example, A. and L. Föppl, Drang and Zwang, Vol. 1, second edition, 1924, p. 302.

$$\sigma_r = \frac{K\sqrt{r}}{4}(-\sin 2.5\,\theta - 3\sin 0.5\,\theta) \quad \text{(4)}$$

$$\sigma_t = \frac{K\sqrt{r}}{4}(\sin 2.5\,\theta - 5\sin 0.5\,\theta) \quad \text{(5)}$$

$$\tau_{rt} = \frac{K\sqrt{r}}{4}(-\cos 2.5\,\theta + \cos 0.5\,\theta) \quad \text{(6)}$$

It is noted that $\sigma_t = \tau_{rt} = 0$ when $\theta = 0$ and $\theta = 2\pi$; that is, the conditions at the crack are satisfied. Equation (2) is satisfied. F and the state of stresses are symmetrical with respect to the line $\theta = \pi$. On the section $\theta = \pi$ one finds $\tau_{rt} = 0$ and the compressive stress

$$s = -[\sigma_t]_{\theta = \pi} = K\sqrt{r} \quad \text{(7)}$$

which will be represented by a parabola with vertical axis, as was indicated by Hertz's theory of pressures of contact.

With z denoting the width of the crack at the distance r from the origin of the coordinates, and E denoting the modulus of elasticity, the curvature of the originally straight line forming one edge of the crack may be written as

$$\frac{1}{2}\,\frac{d^2z}{dr^2} = -\frac{1}{Er}\left[\frac{\partial\sigma_r}{\partial\theta}\right]_{\theta = 0} \quad \text{(8)}$$

Substitution from equation (4) gives

$$\frac{d^2z}{dr^2} = \frac{2K}{E\sqrt{r}} \quad \text{(9)}$$

$$z = \frac{8K}{3E}\,r^{1.5} \quad \text{(10)}$$

Since all the stresses, including σ_r, are zero at the edge of the crack, equation (10) may still be applied when the crack departs moderately from the assumed straight line.

The stress function in equation (3) will be found to represent the most significant features at a major crack in a reinforced concrete beam. To discover additional features the following more general type of stress function will be considered:

$$F = \sum_{1,2,3\ldots}^{n} \frac{K_n\, r^{\,n+1.5}}{2\,(n + 0.5)\,(n + 1.5)}\,[(n - 0.5)\sin(n + 1.5)\,\theta - (n + 1.5)\sin(n - 0.5)\,\theta] \quad \text{(11)}$$

in which $K_1, K_2, \ldots K_n, \ldots$ are constants. This function satisfies equation (2) and the conditions at the crack. With $K_1 = K$ and the remaining constants equal to zero, equation (11) becomes the same as equation (3). The function F in equation (11) is symmetrical with respect to the line $\theta = \pi$. The same process that was applied before gives the compressive stress on the section $\theta = \pi$:

$$s = \sum_{1,2,3\ldots}^{n} (-1)^{n-1} K_n \, r^{\,n-0.5} \qquad (12)$$

and the width of the crack:

$$z = \frac{8}{E} \sum_{1,2,3\ldots}^{n} \frac{K_n}{2n+1} \, r^{\,n+0.5} \qquad (13)$$

An additional term with $n = 0$ would lead to infinite stresses at $r = 0$. A negative value of n would indicate the transmission of infinite total forces directly above the crack, which would be impossible. Therefore, only positive values of n need be considered.

REINFORCED CONCRETE BEAM

Fig. 3 shows a part of a reinforced concrete beam. The cross section is rectangular. The bending moment is assumed to be constant within the part that is shown. Then the total vertical shears will be zero within this part. A vertical crack rises a distance a from the bottom. The study will be limited to this case; inclined cracks, such as may develop with vertical shear, will not be considered here.

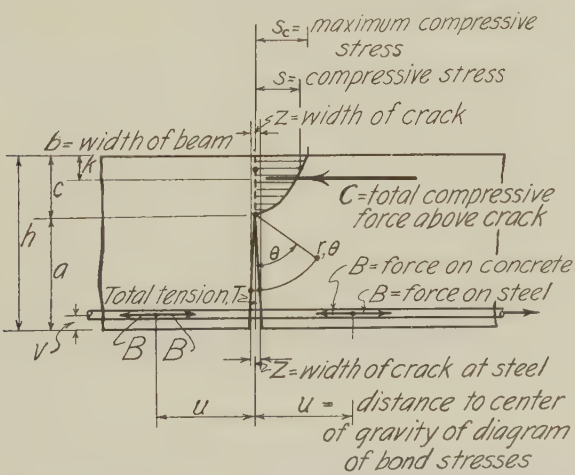

FIG. 3—REINFORCED CONCRETE BEAM WITH RECTANGULAR CROSS SECTION. VERTICAL CRACK AT SECTION WITHOUT VERTICAL SHEAR.

Equations (3) and (10) will define the most essential features of the stresses and deformations around the upper end of the crack. Since the rectangular corners formed at the lower end of the crack are unloaded, it will be required that

$$\left[\frac{d^2z}{dr^2}\right]_{r=a} = 0 \qquad (14)$$

and consequently terms will be needed supplementing equations (10) and (3). One may resort then to the more general functions in equations (11) to (13). Inspection of equation (13) shows that a simple method of satisfying equation (14) is by choosing

$$K_1 = K, \quad K_2 = -\frac{K}{3a}, \quad K_3 = K_4 = \ldots\ldots = 0 \qquad (15)$$

It will be noted that with this choice of constants the first, second, and third derivatives of F with respect to r and θ will be zero at the lower end of the crack, and accordingly this region will be properly free from stresses.

Further supplementary terms will be needed if the stress function is to account for the conditions farther away from the crack. It may be assumed, however, that if the crack extends at least to the middle of the beam, the terms containing K_1 and K_2 will furnish an adequate representation of the conditions at the crack and directly above it, provided that the compressive stresses remain within the proportional limit.

Then equations (12), (13), and (15) lead to the following two formulas:

Compressive stress on the vertical section directly above the crack:

$$s = K\sqrt{r}\left(1 + \frac{r}{3a}\right) \qquad (16)$$

Width of the crack:

$$z = \frac{8K}{3E}\, r^{1.5}\left(1 - \frac{r}{5a}\right) \qquad (17)$$

That the crack should be assumed to extend at least to the middle of the beam is suggested by the observation that equation (16) gives $\dfrac{d^2s}{dr^2} = 0$ when $r = a$.

With the notation shown in Fig. 3 the following further formulas are obtained from equations (16) and (17):

Maximum compressive stress:

$$s_a = K\sqrt{c}\left(1 + \frac{c}{3a}\right) \qquad (18)$$

Total compressive force on the vertical section directly above the crack:

$$C = \frac{2}{3} K b c^{1.5} \left(1 + \frac{c}{5a}\right) \quad \text{(19)}$$

Distance of C below the top of the beam:

$$k = \frac{2c\,(7a + c)}{7\,(5a + c)} \quad \text{(20)}$$

Width of the crack at the steel:

$$Z = \frac{8K}{15Ea} (a - v)^{1.5} (4a + v) \quad \text{(21)}$$

Width of the crack at the bottom:

$$z_a = \frac{32Ka^{1.5}}{15E} \quad \text{(22)}$$

Angular opening of the crack at the bottom:

$$\alpha = \left[\frac{dz}{dr}\right]_{r=a} = \frac{8K\sqrt{a}}{3E} \quad \text{(23)}$$

The possibility of numerical solutions arises from the fact that the width of the crack at the steel, Z, is defined not only by equation (21), but also by the bond resistance of the steel. With D denoting the diameter of the steel bars, A the area of the total cross section of the steel, and f some function, perhaps it may be assumed that

$$\frac{Z}{D} = f\left(\frac{T}{A}, \frac{B}{A}\right) \quad \text{(24)}$$

The determination of the function f for given properties of the steel and the concrete is a clear-cut experimental problem. When the function f is known, it will be possible for each length of the crack, a, to determine a value of K which will lead to the same value of Z by equations (21) and (24).

If the stresses in the steel do not exceed the proportional limit, then the distance u from the crack to the center of gravity of the diagram of bond stresses may be computed approximately as

$$u = \frac{E_s A Z}{2T} \quad \text{(25)}$$

E_s denoting the modulus of elasticity of the steel. If the stresses in the steel exceed the proportional limit, the distance u will be smaller and may be estimated by replacing E_s by an average secant modulus.

When the distance u is known, it becomes possible to estimate the contribution of the crack to the deflections of the beam. Let M denote the bending moment and I the moment of inertia of the compound cross section, including the total area of the concrete plus $\frac{E_s}{E}$ times the

area of the steel. Then, without the crack, the deflections may be computed from the elastic weights $\frac{M}{E\,I}$ per unit of length of the beam. With the crack, the elastic weights $\frac{M}{E\,I}$ within the distance u on each side of the crack will be replaced approximately by the change of slope at the crack at the bottom of the beam. This change of slope is the same as the angular opening α at the bottom of the crack, defined by equation (23). It follows that the additional deflections due to the crack may be computed approximately by assuming an elastic weight at the crack, equal to

$$w = \alpha - \frac{2\,M\,u}{E\,I} \qquad (26)$$

If the crack occurs at the center of a simple span l, the additional deflection at the crack is found as $\frac{1}{4}wl$.

If the compressive stresses above the crack exceed the proportional limit, equation (16) may still be used for the smaller values of r. If the stress at the top reaches the ultimate stress, it will be plausible to modify the diagram for s defined by equation (16) by changing it into a quadrant of an ellipse which retains the radius of curvature at the bottom of the diagram. This ellipse has the equation

$$s = K\sqrt{r - \frac{r^2}{2c}} \qquad (27)$$

which replaces equation (16) in this case. Equation (17) requires no modification. Equations (18) to (20) are replaced as follows:

$$s_c = K\sqrt{\frac{c}{2}}\,, \quad C = \frac{\pi}{\sqrt{32}}K\,b\,c^{1.5}, \quad k = \frac{4\,c}{3\pi} \qquad (28)$$

Equations (21) to (26) require no change.

The quadrant of the ellipse may have to be replaced by the quadrant of some other oval, with the same curvature at the bottom. The corresponding modifications of equations (27) and (28) involve no particular difficulty.

NUMERICAL EXAMPLE

Consider a simple beam with span l = 200 in. and with two equal loads at the third-points. The influence of a vertical crack at the center of the span is to be investigated.

Let b = 10 in., h = 20 in., v = 2 in., A = 2.0 in.^{2}, E = 2,500,000 lb. in.$^{-2}$, E_s = 30,000,000 lb. in.$^{-2}$, ultimate compressive stress of the concrete = 2500 lb. in.$^{-2}$. The compound cross section is found to have its center of gravity 10.79 in. below the top, and its moment of inertia becomes I = 7935 in.^{4}.

Case 1. Assume $a = 14$ in., $c = 6$ in., $s_c = 2000$ lb. in. $^{-2}$. Equations (18) to (20) give

$K = 714.4$ lb. in. $^{-2.5}$, $C = T = 76000$ lb., $k = 2.35$ in. .... (29)

The bending moment becomes

$$M = C\,(h - k - v) = 1{,}189{,}000 \text{ in. lb.} \qquad (30)$$

Equations (21) to (23) give

$Z = 0.0262$ in., $z_a = 0.0319$ in., $\alpha = 0.00285$ .... (31)

Under the assumption that the stresses in the steel do not exceed the proportional limit, equation (25) gives $u = 10.34$ in. Then, by equation (26) one finds $w = 0.00161$. The corresponding diagram of additional deflections contributed by the crack is a triangle with the altitude $\frac{1}{4} \cdot 0.00161 \cdot 200$ in. $= 0.0805$ in. If the beam is imagined to have no cracks, one finds the maximum deflection

$$\delta = \frac{23\,M\,l^2}{216\,E\,I} = 0.2553 \text{ in.} \qquad (32)$$

The particular crack which has been studied adds 32 per cent to this deflection. Additional cracks must be expected under the circumstances, but the present problem is limited to the study of the one crack at the center of the span.

Case 2. If the maximum stress in the steel is close to the yield point in Case 1, a moderate increase of the load may cause the steel to yield and the crack to grow.

Assume $a = 16$ in. , $c = 4$ in. , $s_c = 2500$ lb. in. $^{-2}$.

Since the ultimate stress of the concrete has been reached, equations (28) will be used instead of equations (18) to (20). One finds

$K = 1768$ lb. in. $^{-2.5}$, $C = T = 78540$ lb.,

$k = 1.70$ in., $M = 1{,}280{,}000$ in. lb. .... (33)

Equations (21) to (23) give

$Z = 0.0815$ in., $z_a = 0.0966$ in., $\alpha = 0.00754$ .... (34)

Equation (25) cannot be used because the steel has begun to yield. Assuming $u = 10.34$ in. as in Case 1, one finds, by equation (26), $w = 0.00621$. The contribution of the crack to the maximum deflection becomes $\frac{1}{4}\,wl = 0.3105$ in. Without any cracks the maximum deflection would be 0.2748 in. The crack has added 113 per cent to the deflection.

CONCLUDING REMARKS

The analysis which has been presented draws on the theory of elasticity to furnish certain features of the picture, but the treatment is not a complete solution by the mathematical theory of elasticity. Judgment has been applied in deciding what is important and what may be ignored. Furthermore, the considerations of stresses beyond the proportional limit and of the slipping of the steel are outside the present province of the mathematical theory of elasticity. Therefore, the analysis is not exact, but it may serve a purpose. Some theory, supplying a picture of the action, is always used in planning, making,

and interpreting a test. It is not detrimental if some of the features of this picture are distorted, because observations may correct the distortions; but it is disadvantageous if some of the important features are not shown at all. The analysis which has been presented includes some features which are not considered in the conventional theory.

The theory presented suggests a questioning of the rationality of nominal working stresses which are independent of the relative size and amount of the steel.

For such discussion of this paper as may develop readers are referred to the JOURNAL *for May-June, 1934. Discussions should be available to the Secretary by April 1, 1934.*

A New Type of Dam*

BY A. C. JANNI†

Since Pelletrau, in 1897, suggested the arch shape instead of straight line gravity dam, that form of dam is being widely used.

In view of the expensive, illogical and unscientific nature of the straight line gravity dam, it is not surprising that this new shape, being comparatively logical and economical, should supplant the other in so short a time.

However, although the solid arch dam, as generally built at present, permits, on the basis of certain assumptions, of a better analysis, yet some of those assumptions are known not to be correct.

The law of distribution of stresses in the body of those large masses of concrete is somewhat uncertain, and this condition is still more emphasized by the fact that the mass of concrete, constituting the dam, is made up of sections all differing from each other, as to their age, in other words, the placticity of the concrete is not uniform throughout the structure.

While this condition may be somewhat advantageous from the point of view of stability of the dam, it creates, nevertheless, a condition of equilibrium quite different from the one obtained by the analysis.

The temperature stresses, which constitute the largest part of the total stresses, cannot be determined with sufficient accuracy. When to this uncertainty is added that deriving from the temperature due to setting, and which in some dams can last years, and it is not uniform, as it has been found, the solution of this problem becomes very much involved.

Nor can the use of models or templates help the analysis, for the deformations due to temperature changes are of two distinct kinds; the first deforms the structure, therefore stresses can be computed; the second, does not deform the structure, yet analyses show that its effect is important, and cannot be disregarded.

The danger of uplifting, due to infiltration of water under pressure below the foundations of the dam, is always present in every dam of this kind.

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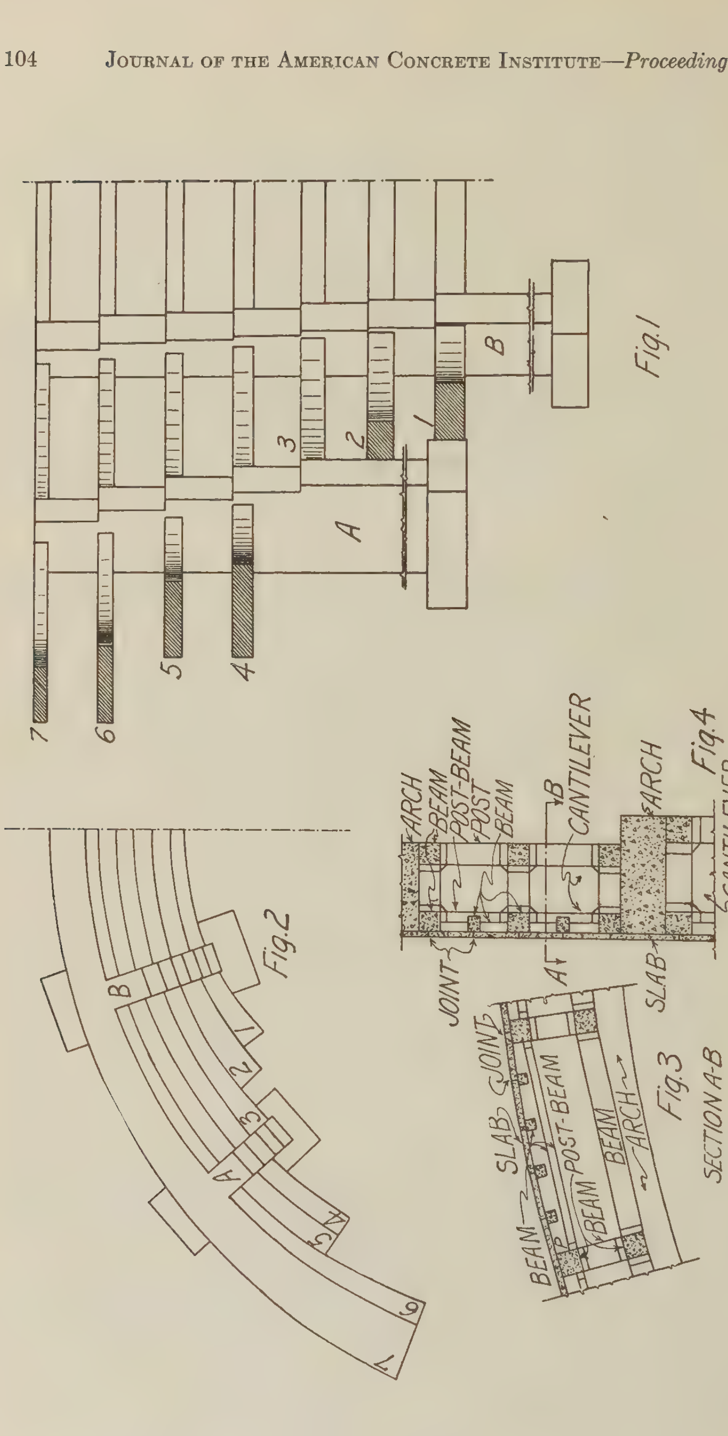

FIG. 1, 2, 3, 4—NEW TYPE OF DAM

Fig. 1—Elevation without posts and beams. Fig. 2—Plan of half dam. Fig. 3—Section A-B Fig. 4—Method of connecting posts and beams with arches and cantilevers

The eventual formation of vertical cracks, parallel, in a general way, to the face of the dam, is another possibility which cannot be disregarded, especially if a crack of that nature happens to be connected with some horizontal crack originating from the water face of the dam.

Another cause of danger is the formation of horizontal cracks in the body of the dam. Upon their uplifting action there is a decided uncertainty; writers on this subject have not agreed upon the amount of uplifting force at the given level of a crack, and the designer is compelled to make some assumptions which are in open contrast with the principles of equilibrium of a solid.

Furthermore, the law of distribution of the hydrostatic pressure on the back of a solid dam, be it straight or arched dam, due to its monolithic nature, is not clear.

By theoretical considerations, as well as by tests, it has been demonstrated that the hydrostatic pressure against any one of the imaginary horizontal rings, into which, for the purpose of analysis, the dam is supposed to be divided, is not uniform at all horizontal points of that ring; but the law of that variation is not known.

Breville, R. Schaefer, Le Rond, Levy, Guidi, Mohr, etc., have expressed their opinion on the serious uncertainties existing in the assumptions followed in designing solid dams; the last two eminent scientists have come to the conclusion that, given the several uncertainties, creeping right along with the assumptions, it is not to be expected that the results given, even by the most modern methods of analysis, will correspond to the actual stresses.

In conclusion, while the engineer has at his disposal very modern methods of analysis, by which he should be able to determine the stresses in a dam with great accuracy, the assumptions he is compelled to make, in order to carry out the analysis, are such as to make the results almost illusory.

The new type of dam presented here has been evolved having in mind what seemed to the writer the best way of avoiding practically all the uncertainties existing in a structure of the type mentioned.

Fig. 1, 2, 3 and 4 show this dam in its essential parts. The main frame consists of a series of arches, supported at various points by cantilevers; each arch butts against solid rock at each end, independently from each other; also each cantilever can be made to reach solid rock independently of each other.

Two systems of beams, one horizontal the other vertical, run in each panel formed by two consecutive arches and two consecutive cantilevers.

Slabs, independent from each other, are cast against those beams. The systems of beams and slabs is to extend below grade to a convenient depth.

This is a general outline of this new type of dam, the details of which are subject to variations suggested by the physical conditions of the site.

From a practical point of view, the reader will soon realize that a dam of this type offers not only a very material economy in cost and time, but also, comparatively, a simpler method of erection.

From the scientific point of view, the analysis of this structure does not present anything altogether new, or of difficult application.

The slabs, as well as the beams, can be computed for any given levels as carrying the hydrostatic pressure at that level, and having in mind the absence of continuity in each slab.

The skeleton of the dam, made up of arches and cantilevers, constitutes, of course, an hyperstatic system.

The limits imposed upon this writing do not permit showing the method of analysis in full; therefore, a brief outline will be given here.

The fundamental principle underlying this analysis is that the deflection of each cross point between an arch and a cantilever is to be the same.

This basic consideration enables the engineer to write as many systems of simultaneous first degree equations, as there are cantilevers in the structure; each system having as many equations as there are cross points between the arches and the particular cantilever considered.

If the structure is symmetrical, then the number of systems is reduced by one half.

In the case shown in the figure, the engineer has only two systems of simultaneous equations, one with seven and another with five equations.

The solution of those systems of equations will give the values of that part of the hydrostatic pressure actually acting upon the cantilever at the various cross points.

Once these values are known, the values of the hydrostatic pressures actually acting upon the arches at those same points is easily computed; therefore, the analysis of each arch and each cantilever can be carried out independently.

The above method, strictly correct in itself, does not differ materially from the method used by engineers in the analysis of a solid arch dam; but the assumptions made, in this last case, are such as to vitiate the

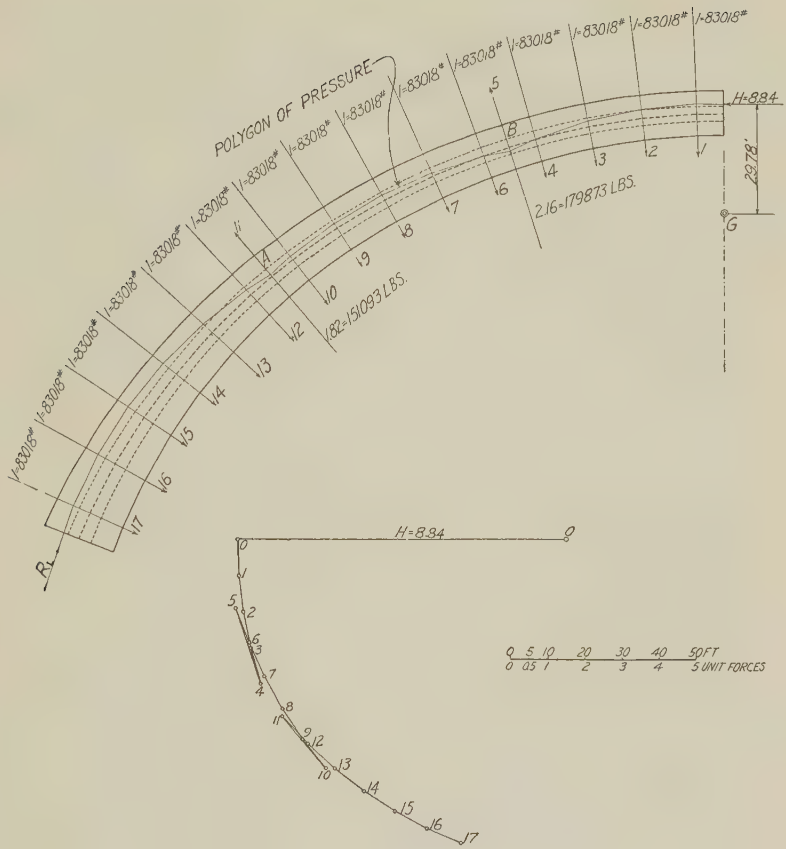

Fig. 5

results obtained by the analysis. The fault is neither with the engineers, nor with the structure, but with experience which, so far, has failed to ascertain the dependability of some data which are so important in establishing assumptions.

The dam presented here has been analyzed in its main parts, after being drawn to scale. It may be interesting to the reader to examine the behavior of one of the arches when its deformation, under the action of hydrostatic pressure and temperature, is contrasted by the presence of cantilevers.

Fig. 5 shows the top arch 7, the dimensions of which are:

Radii	Extrados	201 ft.
	Intrados	180.5 and 168.5 ft.

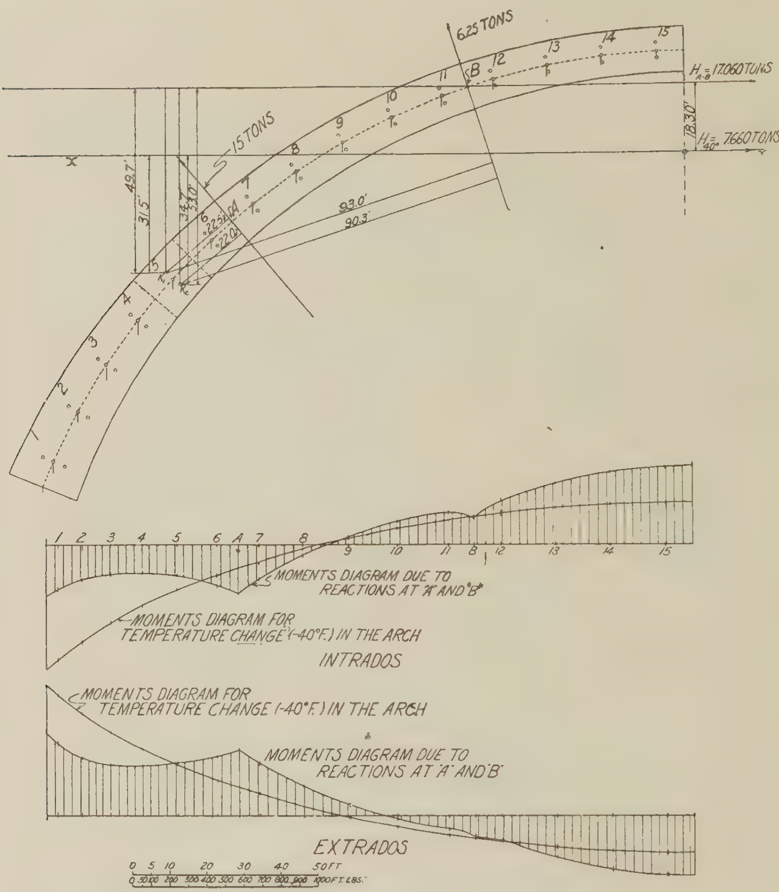

FIG. 6

Cross Section	Crown	1 x 12 sq. ft.
	Springing	1 x 20 sq. ft.

This figure shows the polygon of pressure due to the hydrostatic pressure.

Fig. 6 shows the moment diagrams due to a lowering of 40°F. The shaded part of those diagrams represents the algebraic summation of the moments caused in the arch alone, and of the moments due to the presence of cantilevers.

The method used for finding the moments due to the reactions at A and B is shown in full for voussoir 5.

The tendency to some side deflection and torsion in each cantilever can be easily taken care of with the methods offered by analysis.

In connection with the torsion, it must be pointed out that its effects upon the cantilever are, so to speak, of a secondary importance when compared with the stresses induced by deflection.

It is well known that the stresses due to torsion, in a prismatic solid, decrease from the center of the cross section of that prism to the edges of it, therefore those maximum stresses are located in that part of the cantilever where the stresses due to its deflection are a minimum.

For such discussion of this paper as may develop readers are referred to the JOURNAL *for May-June, 1934. Discussions should be available to the Secretary by April 1, 1934.*

Simplified Concrete Mix Design*

BY H. N. WALSH†
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SUMMARY

1. Certain gradings of aggregates and cement that produce dense, workable, and strong concrete have been determined.

2. From these, by eliminating the cement, the author has determined gradings of aggregates alone which are suitable for combination with different proportions of cement so as to yield dense, workable and relatively strong concrete.

3. Four diagrams are given showing the author's recommended gradings: Fig. 1 is for gravel aggregates of ¾ in. maximum size and mixes ranging from one part cement to four parts mixed aggregate by loose volume to 1:9; Fig. 2 is for crushed stone of ¾ in. maximum size and mixes ranging from 1:3 to 1:8; Fig. 3 is for gravel aggregates of 1½ in. maximum size and mixes ranging from 1:3½ to 1:9; and Fig. 4 is for crushed stone aggregates of 1½ in. maximum size and mixes ranging from 1:3 to 1:8.

4. Three methods are given for determining the proportion of cement to mixed aggregate to be used. A diagram is included showing the relation between the cement content of the finished concrete and the proportion of cement to mixed aggregate.

5. An example of the use of the method is worked out fully.

6. Notes on precautions and allowable variations from the recommended gradings are given.

7. References.

In recent years European engineers have investigated the effects of grading on the properties of concrete, and developed the use of grading curves for the design of mixes. Their researches have definitely established the type of grading that produces high quality concrete, which has for its characteristics workability when freshly mixed, density when placed, suitable cement content, and adequate strength when cured. The results of these researches furnish the basis for a

*Received by the Secretary September 9, 1933.
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method of designing mixes which is simple in its application, and which can be used without extensive changes in specifications or inconvenience to contractors.

DENSITY AND WORKABILITY

The approach to a theory of concrete mixes can be simplified by taking no account of strength, ignoring the active properties of the cement by regarding it temporarily as an inert powder, and concentrating on the idea of a mixture of graded solid particles and water. This mixture should be sufficiently workable to be placed satisfactorily by the methods proposed: stiff and bordering on harshness when it is to be in large masses and heavily rammed or vibrated, or unctuous and capable of flowing without segregation when it is to be placed between and around complicated reinforcement in narrow forms. When placed and set, it must be dense or compact.

For ordinary purposes and with materials of average specific gravity, weight per cubic foot is a sufficient index of density.

The mixture when placed in a mold will weigh more or less per cubic foot according to the way in which the particles are graded. For a fixed maximum particle size one particular grading will give a greater weight per cubic foot than any other. But the mixture so graded to give maximum density will be harsh and difficult to place, especially if the biggest particles in it are more than one inch in diameter, or are angular. However, by suitably increasing the proportions of medium and fine particles, sufficient workability for easy placing can be gained without serious reduction of density.

Better workability can be obtained by using more fines. But this further improvement in workability is attended by a rapid decrease in density. Therefore, the grading to be aimed at is that which just produces a workable mix while preserving a density only slightly less than the maximum possible.

In this mixture the finest particles are cement. The proportion of fines is fixed for a given maximum size of particle and the ideal grading for density combined with workability. Changing the proportion of cement involves changing the proportion of fines in the aggregate to maintain as nearly as possible the same grading for the mix. On reducing the cement the fines in the aggregate must be increased, and vice versa. But some alteration of the other sizes is also necessary. In this way the grading of the mix may be retained more or less constant while the proportion of cement is varied.

Feret, Fuller, Bolomey[1], Dutron[2], Furnas and Anderegg[3] have determined gradings for the complete mix, which produce both density

[1], [2], [3] See references at end of paper.

and workability. From their curves, formulae and tables, it is possible to derive for any proportion of cement and maximum particle size the grading of the aggregate alone that produces the ideal mix. The next step is to select the proportions in which to mix the available fine and coarse aggregates, so that the mixed aggregate may have the derived grading. Then, the final mix of cement, aggregate, and water, will be dense and workable.

STRENGTH

Dutron[2] has shown that with a fixed cement content the grading which produces the strongest concrete is nearly the same as that for maximum density, other things being constant. He has shown also that strength is only slightly reduced by the addition to the maximum density grading of those proportions of medium and fine particles which render the mix workable without seriously reducing the density. Therefore, the grading for density combined with workability is also a strength producing grading.

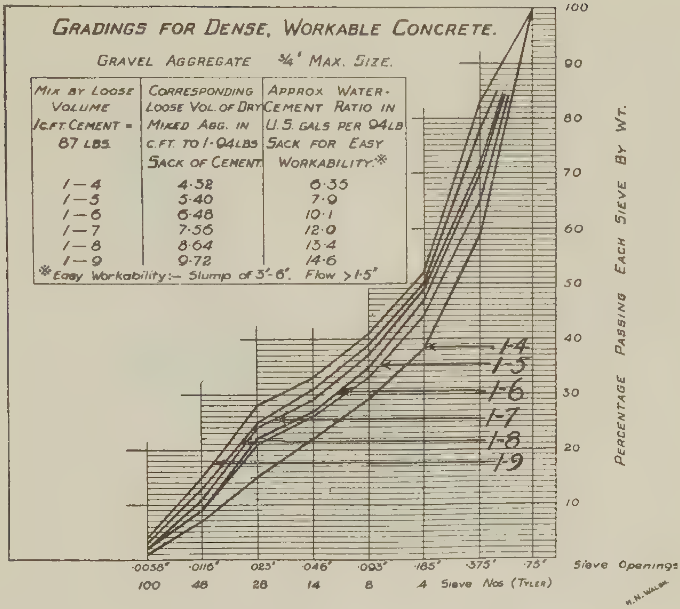
This means that the grading giving high density combined with workability requires a relatively small proportion of water to produce the consistency ordinarily used. Therefore, the water-cement ratio will be lower with this grading than with any other grading that produces equally workable concrete of the same cement content and consistency. If the mix has more fine aggregate, more water will be required to produce the same consistency, and the concrete will be weaker. If it has less fines, the concrete may require less water, but it will be harsher and insufficiently workable.

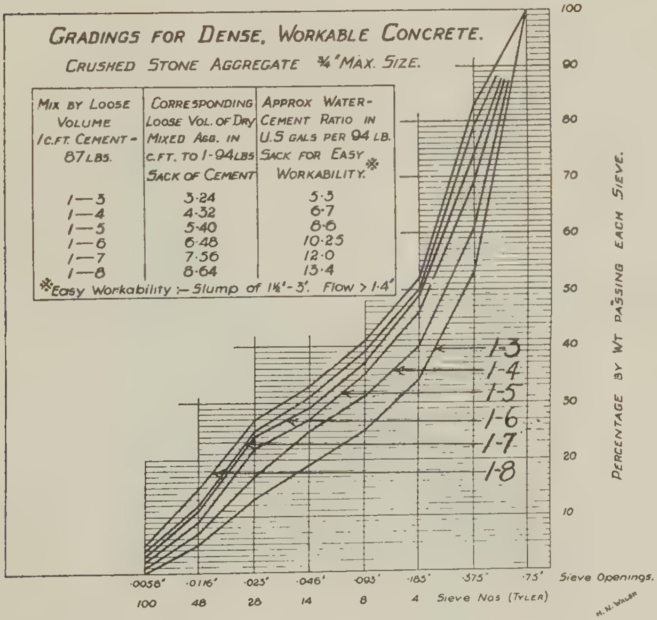
RECOMMENDED AGGREGATE GRADINGS

Gradings of aggregates alone suitable for combination with different proportions of cement have been determined by the writer, and published in tabular form in the Trans. of the Inst. of Civil Engineers of Ireland (May, 1933, Vol. LIX).[4] Different gradings are necessary for gravel and crushed stone aggregates. So far the writer's work has been completed only for aggregates of ¾ in. and 1½ in. maximum particle size. These gradings are reproduced in diagram form in Fig. 1 to 4, the mix for which each grading is suitable being indicated on the curve.

Fig. 1 is for gravel aggregates of ¾ in. maximum size, and for mixes ranging from 1 part cement by loose volume to 4 parts of mixed aggregate by loose volume to 1:9. Fig. 2 is for crushed stone aggregates of the same size, and for mixes ranging from 1:3 to 1:8. Fig. 3 is for gravel aggregates of 1½ in. maximum particle size and mixes ranging

[4]Reviewed by J. C. Pearson, *News Letter*, p. 5, JOURNAL AMER. CONCRETE INST., Sept.-Oct. 1933.


GRADINGS FOR DENSE, WORKABLE CONCRETE.
GRAVEL AGGREGATE ¾' MAX. SIZE.
MIX BY LOOSE VOLUME 1C.FT. CEMENT = 87 LBS.
CORRESPONDING LOOSE VOL. OF DRY MIXED AGG. IN C.FT. TO 1-94LBS SACK OF CEMENT.
APPROX WATER-CEMENT RATIO IN U.S. GALS PER 94LB SACK FOR EASY WORKABILITY.*
1—4 4·32 6·35
1—5 5·40 7·9
1—6 6·48 10·1
1—7 7·56 12·0
1—8 8·64 13·4
1—9 9·72 14·6
* Easy Workability:— Slump of 3'-6'. Flow >1·5'
1-4
1-5
1-6
1-7
1-8
1-9
100
90
80
70
60
50
40
30
20
10
PERCENTAGE PASSING EACH SIEVE BY WT.
·0058' ·0116' ·023' ·046' ·093' ·185' ·375' ·75' Sieve Openings
100 48 28 14 8 4 Sieve Nos (TYLER)
H.N. WALSH


GRADINGS FOR DENSE, WORKABLE CONCRETE.
CRUSHED STONE AGGREGATE ¾' MAX. SIZE.
MIX BY LOOSE VOLUME 1C.FT. CEMENT - 87LBS.
CORRESPONDING LOOSE VOL. OF DRY MIXED AGG. IN C.FT. TO 1-94LBS SACK OF CEMENT
APPROX WATER-CEMENT RATIO IN U.S GALS PER 94 LB. SACK FOR EASY WORKABILITY.*
1—3 3·24 5·3
1—4 4·32 6·7
1—5 5·40 8·6
1—6 6·48 10·25
1—7 7·56 12·0
1—8 8·64 13·4
*Easy Workability :— Slump of 1½'-3'. Flow >1·4'
1-3
1-4
1-5
1-6
1-7
1-8
100
90
80
70
60
50
40
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10
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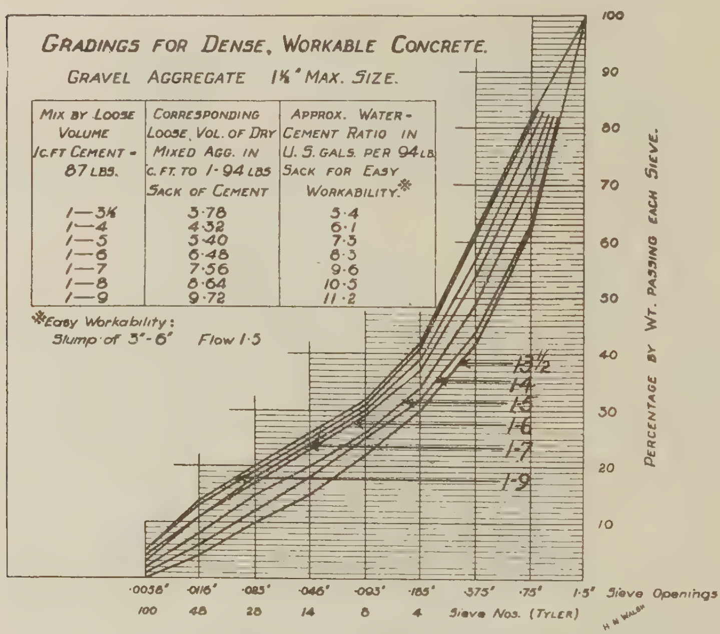
100 48 28 14 8 4 Sieve Nos (TYLER)
H.N. WALSH

FIG. 1, 2


GRADINGS FOR DENSE, WORKABLE CONCRETE.
GRAVEL AGGREGATE 1½" MAX. SIZE.
MIX BY LOOSE VOLUME 1C.FT CEMENT = 87 LBS.
CORRESPONDING LOOSE VOL. OF DRY MIXED AGG. IN C.FT. TO 1·94 LBS SACK OF CEMENT
APPROX. WATER-CEMENT RATIO IN U.S. GALS. PER 94 LB SACK FOR EASY WORKABILITY.*
1—3½ 3·78 5·4
1—4 4·32 6·1
1—5 5·40 7·5
1—6 6·48 8·5
1—7 7·56 9·6
1—8 8·64 10·5
1—9 9·72 11·2
*Easy Workability: Slump of 3"-6" Flow 1·5
1-3½
1-4
1-5
1-6
1-7
1-9
100
90
80
70
60
50
40
30
20
10
PERCENTAGE BY WT. PASSING EACH SIEVE.
·0058" ·0116" ·023" ·046" ·093" ·185" ·375" ·75" 1·5" Sieve Openings
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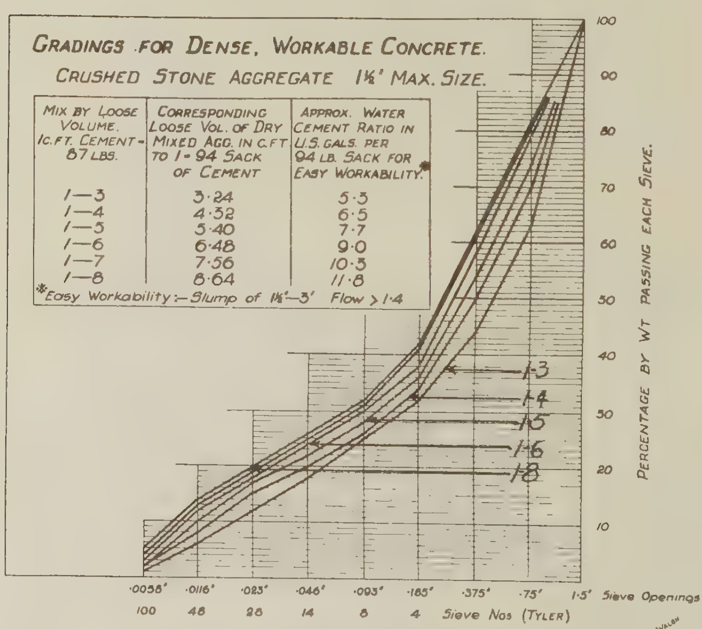
H N WALSH


GRADINGS FOR DENSE, WORKABLE CONCRETE.
CRUSHED STONE AGGREGATE 1½" MAX. SIZE.
MIX BY LOOSE VOLUME. 1C.FT. CEMENT = 87 LBS.
CORRESPONDING LOOSE VOL. OF DRY MIXED AGG. IN C.FT TO 1·94 SACK OF CEMENT
APPROX. WATER CEMENT RATIO IN U.S. GALS. PER 94 LB. SACK FOR EASY WORKABILITY.*
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1-3
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PERCENTAGE BY WT PASSING EACH SIEVE.
·0058" ·0116" ·023" ·046" ·093" ·185" ·375" ·75" 1·5" Sieve Openings
100 48 28 14 8 4 Sieve Nos (TYLER)
H N WALSH

FIG. 3, 4

from 1:3½ to 1:9. Fig. 4 is for crushed stone aggregates of the same maximum size and mixes ranging from 1:3 to 1:8. A cubic foot of cement loose is taken as weighing 87 lbs., which was its weight as obtained by the methods of filling measures employed in the writer's laboratory. The corresponding volumes of mixed aggregate (dry and loose) to be used with 94 lbs. of cement are tabulated on the diagrams.

Each curve on each diagram shows the coarsest grading that will make a *dense and workable* concrete with the proportion of cement stated. If a coarser grading is used the concrete will be harsher. If a finer grading is used the concrete will lose density.

Conversely, the proportion of cement to mixed aggregate recommended for each grading is the least proportion of cement (leanest mix) that will make an easily workable concrete with aggregate of that grading. With less cement the concrete will require tamping. With more cement the concrete will be more workable, but will lose density.

Any aggregate having a grading that is averaged by one of the curves, and nowhere departs very far from that curve, may be used with the proportion of cement stated, and will make a concrete that is dense, easily workable, and relatively strong for the proportion of cement used, provided, of course, that the aggregate itself is sound.

All aggregate volumes are for *dry* materials filled into the measures without rodding or tamping or shaking. With damp aggregates allowances must be made for bulking. This is very important, especially for the gradings containing high proportions of fines.

The water-cement ratios stated on the diagrams are somewhat high, especially for the leaner mixes, and should be used with caution. The consistency and workability are suitable for use with complicated reinforcement, and lean mixes for plain concrete could be made much drier. Much less water than is recommended should be used in the first batch, and gradually increased until the required consistency is reached.

For convenience each curve is called a "type-curve" or "type-grading," and these terms may be regarded as synonymous. Thus, the curve marked 1:4 on Fig. 1 gives the type-grading for use in a 1:4 mix with gravel aggregate of ¾ in. maximum size.

The sieve sizes shown on the diagrams are those of Tyler's sieves No. 100, 48, 28, 14, 8, 4, ⅜ in., ¾ in. and 1½ in. These are the sieves used by the writer.

DESIGNING MIXES

The use of these diagrams makes the design of mixes very simple. To remove danger of misunderstanding it is as well to state that the method to be described does not involve the use of artificial gradings.

Ordinary conditions of having a fine aggregate passing, say, a No. 4 (Tyler) sieve and a coarse aggregate retained on a No. 4 sieve with a little overlap in each are assumed. The diagrams may be used equally well for a bank gravel which is to be separated into two sizes and recombined, or for determining the proportions in which a bank gravel containing some fines and a sand should be combined. In any of these cases the diagrams can be used readily to determine:

1. Whether the fine and coarse aggregate can be so combined as to yield a mixed aggregate suitably graded for making a dense, workable, and strong concrete with the intended proportion of cement.

2. If so, in what proportions the fine and coarse should be combined.

The first step in designing a mix is the fixing of the proportion of cement to mixed aggregate.

The following are three methods of doing so:

A. Those who are accustomed to specifying mixes by arbitrary proportions of the 1:n:2n type may base their proportion of cement to mixed aggregate on the following data:

Cement—Fine Agg.—Coarse Agg.		Cement-Mixed Agg.
1:1½:3	is equivalent to	1 :3.8
1:2 :4	is equivalent to	1 :4.9
1:2½:5	is equivalent to	1 :6.0
1:3 :6	is equivalent to	1 :7.4

That is, where, say, 1:2:4 would have been specified, 1:4.9 is now to be specified, or 94 lbs. cement to 5.63 cu. ft. mixed aggregate (*dry and loose*). A tolerance of ± 0.1 parts mixed aggregate may be allowed.

B. Bolomey has shown that under ordinary conditions the relation of strength to cement-water ratio by weight is expressed with sufficient accuracy by the straight line formula:

$S = K\ (c/w - 0.5)$,
where c/w = ratio by weight of cement to water,
and K = a strength factor depending on the cement, curing conditions, and age of the concrete.

With a rapid hardening cement used by the writer, and concrete cured in water and tested at 7 days K was 3500 p. s. i. Actual test results under job conditions will generally be within ± 20 per cent of the strengths calculated by the above formula. In a particular case the value of K must be determined for the cement used and the other conditions prevailing. The value of c/w corresponding to the required strength is then calculated and converted to gallons per sack. The mix corresponding to this water-cement ratio is then taken from the appropriate table, and the corresponding aggregate grading is used.

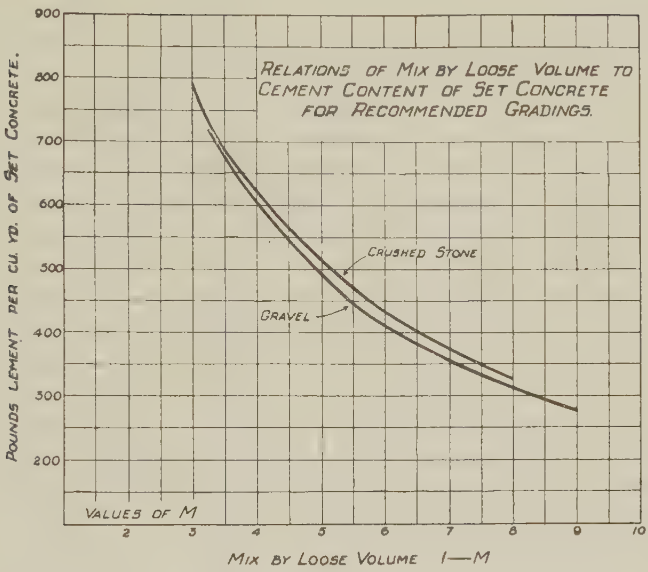

Fig. 5

C. A third method of specifying mixes is by the weight of cement per cubic yard of finished concrete. It is not possible to state an exact relationship between the proportions of cement to mixed aggregate and the cement content of the finished concrete. However, Fig. 5 shows to within 3 per cent the weight of cement per cubic yard of finished concrete for mixes proportioned 1 part cement to *M* parts mixed aggregate by loose volume, provided the grading of the aggregate corresponds approximately to the corresponding type-grading. When the cement content is specified in lb. per cu. yd. of hardened concrete the proportion of cement to mixed aggregate can be read from this diagram, and the corresponding grading used.

PROPORTIONING THE AGGREGATES

When the type-grading has been selected by one of the above methods, the next step is to determine the proportions in which the available aggregates should be combined so that the grading of the mixed aggregate will agree as closely as possible with the type-grading selected. Granulometric analyses of the aggregates must be made. But it is not necessary that sieves of the openings shown on the diagrams should be used. Any set of sieves having openings uniformly

spaced over the range from about 0.006 in. to the maximum size of the aggregate will be satisfactory.

When the available fine and coarse aggregates are suitably graded it is easy to determine the proportions in which to combine them so that the grading of the mixed aggregate is very nearly the same as the type-grading for the specified mix. If the aggregates are unsuitable no combination of them will have a grading approximating the type grading. In this case other aggregates must be sought or an inferior concrete used.

The procedure in proportioning the aggregates is a simple arithmetical calculation, which will be most easily understood from an example.

Example:

The finished concrete is to contain 600 lb. of cement per cu. yd. The aggregates are a natural sand weighing 97 lb. per cu. ft. and a gravel of ¾ in. maximum size, weighing 90 lb. per cu. ft., the materials being loose and dry in both cases. The lower curve of Fig. 5 shows that the cement content specified corresponds closely to a 1:4 mix. Therefore, the type grading on Fig. 1 for a 1:4 mix is to be used.

The granulometric analyses of the aggregates are given in the first and second lines of Table 1.

TABLE 1

MATERIAL	SIEVE NUMBERS								
	100	48	28	14	8	4	⅜ in.	¾ in.	
	Percentage by Weight Passing each Sieve								
Proposed Sand	4	22+	40	65	80	98	100	100	A
Proposed Gravel:	0	0	0	0	0	4	40	100	B
Type Grading to be Used:	1	7	15	22	29	38	59	100	C
	Actual Weight Passing each Sieve								
36-lb. of Proposed Sand:	1.4	7.9	14.4	23.4	28.8	35.3	36	36	D
64-lb. of Proposed Gravel:	0	0	0	0	0	2.6	25.6	64	E
	Percentage by Weight Passing each Sieve								
Mix of 36-lb. Sand and 64-lb. Gravel:	1.4	7.9	14.4	23.4	28.8	37.9	61.6	100	F

Line C of Table 1 is the type grading for sand and gravel aggregates for a 1:4 mix as taken from Fig. 1. From the analysis of the aggregates and the type grading it seems probable that a mix of 36-lb. sand and 64-lb. gravel would have a grading nearly the same as the type grading. The analysis of 36-lb. of the sand given in line D is got by multiplying the figures in line A by 0.36. Similarly the figures in line E are got by multiplying those in line B by 0.64. The figures in line F, which give the grading got by mixing 36-lb. of the sand with 64-lb. of the gravel, are found by

adding the figures in the same vertical columns in lines D and E. It will be seen that the grading F corresponds closely with the type grading C. This combination of 36-lb. of the sand with 64-lb. of the gravel is, therefore, very suitable for use in a 1:4 mix. The weight of the mixed aggregate when dry and loosely filled into a measure proved to be 112 lb. per cu. ft. (sp. gr. of particles = 2.65).

The proportions of the mix are:

87-lbs. cement to 448-lb. dry mixed aggregate, or

94-lbs. cement to 484-lb. dry mixed aggregate, which is equivalent to

94-lbs. cement to 174-lb. dry sand to 310-lb. dry gravel, or one bag cement to 1.79 cu. ft. loose dry sand, to 3.44 cu. ft. loose, dry gravel, with 6.35 gallons of water per bag.

With sand and gravel particles having a specific gravity of 2.65, concrete of this mix weighed 152 lb. per cu. ft.

On the job the sand would be wet or damp, and suitable allowance should be made for bulking; otherwise the mix would contain too little sand.

The aggregates chosen for the above example were suitably graded. Had the sand or the gravel, or both been unsuitably graded, it would have been impossible to get any combination of them that would have had a grading approximating to the type grading for a 1:4 mix. The limits to which the grading curve of the mixed aggregate may be allowed to depart from the type-curve are important. For general purposes it is sufficient to keep all ordinates of the aggregate grading curve within ± 2 per cent of the type curve, except near the extreme ends of the curves, where wider variations are allowable. At the same time minus variations in one part should be roughly balanced by plus variations in an adjacent loop of the curve. That is, the aggregate curve should be roughly averaged by the type-curve. This applies especially to the parts of the curves lying between sieve openings of 0.023 in. and 0.185 in. for ¾-in. aggregates, and between 0.023 in. and ⅜ in. for 1½-in. aggregates. In doubtful cases or when workability is especially important it is advisable to keep the aggregate curve above the type curve for sieve sizes less than 0.185 in. If methods of placing allow harsher concrete to be used, the aggregate curve may be allowed to lie on the average one to two per cent below the type-curve.

Trial Mix

In all cases a trial mix should be made to ensure that the concrete has the required properties. A small adjustment of the proportions may be necessary, but the second trial should be satisfactory.

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For such discussion of this paper as may develop readers are referred to the JOURNAL *for May-June, 1934. Discussions should be available to the Secretary by April 1, 1934.*

Durability Studies of Concrete and Aggregates*

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I. INTRODUCTION

The purpose of this investigation was to study the durability and strength qualities of different concrete aggregates.

A total of ten aggregates was included in the test program, six natural sands and gravels, three types of crushed limestone, and one crushed iron ore and steel punchings.

II. SUMMARY

(1) For a given type and gradation of aggregates the placibility of the concrete was nearly the same for the three mixes which had a constant water content per cubic yard of concrete.

(2) In general, the strength of the concrete increased in direct proportion to the increase in the cement-water ratio of its paste regardless of the type of aggregate used.

(3) A fair agreement was found between the results of the sodium sulphate tests and the freezing and thawing tests of the fine aggregates.

(4) The length of moist curing at early ages had a marked effect upon the durability of the concrete.

(5) The relation between durability and cement-water ratio was similar to the relation between strength and cement-water ratio.

(6) A fair correlation was found between the durability tests of the aggregates and of the concrete.

(7) The natural sand and gravel aggregates produced a more durable concrete than did the limestone aggregates included in this investigation.

(8) The durability of concrete is an economic problem and can be obtained by the use of high grade aggregate in good quality paste, or by a less high grade aggregate in high quality paste, or by use of low quality paste with long curing or high quality paste with short curing.

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III. DESCRIPTION OF AGGREGATES

For comparison the aggregates are designated by numbers. Aggregates No. 1, 5, 6, are crushed unwashed limestone. These aggregates required a high water content for the production of a placeable mix.

Aggregates No. 2, 3, 4, 7, 8 and 9 were natural washed sands and gravels which are being used extensively in concrete construction.

Aggregate No. 10 consisted of crushed Swedish iron ore as fine material and steel punchings as coarse material. This aggregate produces concrete of a very great weight and was included in this series for the sake of studying its corrosion in concrete.

IV. TEST PROGRAM

Concretes with three different cement-water ratios, 1.25, 1.61, and 2.25 by weight, were made with each kind of aggregate. Three lengths of moist curing were used; 3, 7 and 28 days. Compressive tests were made and freezing and thawing tests started at each of the three ages of moist curing. Three cylinders were made for each type of test, making a total of 1080 cylinders for the investigation. Sodium sulphate tests were also made on concrete cylinders. The following table gives the program of tests of concrete specimens.

TABLE 1—PROGRAM FOR CONCRETE TESTS
Cement-Water Ratios of 2.25, 1.61 and 1.25 by Weight

Age at Test	Comp.	F and T	Sod. Sul.	Add. Comp.	Total No. of Cylinders
3 days	3	3	3	3	12
7 days	3	3	3	3	12
28 days	3	3	3	3	12
					Total 36

The qualities of the aggregates themselves were studied by means of sodium sulphate tests and freezing and thawing tests.

V. PROCEDURE

All concrete mixes had a fine-coarse ratio of 40 to 60 for the aggregates. The cement was donated by the Lehigh Portland Cement Co. and had been stored for more than one year in the laboratory. All concrete specimens were 3 by 6-in. cylinders. A 2- to 5-in.slump was aimed at for all mixes and standard procedure of making and curing was followed.

The freezing and thawing experiments were carried out in submerged condition. One complete cycle of freezing and thawing was obtained in one day.

The sodium sulphate tests on concrete cylinders were carried out at 21°C in a saturated solution for 24 hours, followed by 24 hours of oven

drying at a temperature of 100°C. The cylinders were tested to failure in compression at the end of five cycles of sodium sulphate testing.

The sodium sulphate tests on the aggregates were carried out in the same manner as for the concrete. However, the number of cycles was increased to fifteen. Fifteen cycles of freezing and thawing were also used for the aggregates in order to give a direct comparison between the effects of the two different types of test.

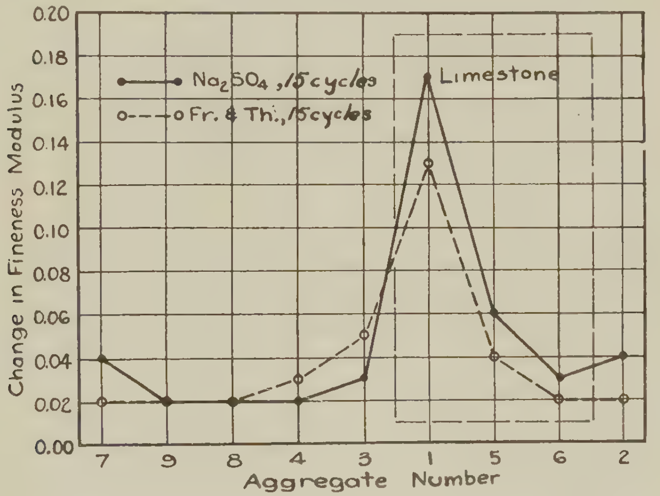


FIG. 1—RESULTS OF SODIUM SULPHATE AND FREEZING AND THAWING TESTS OF NINE DIFFERENT AGGREGATES

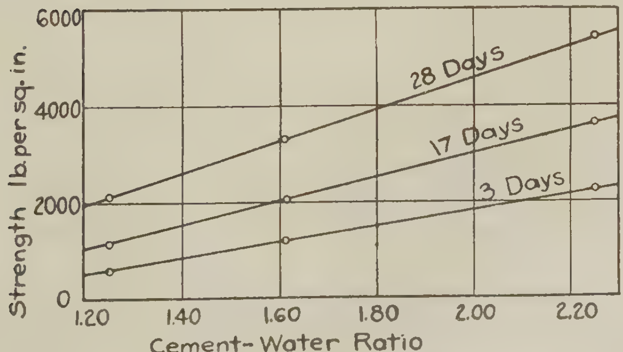
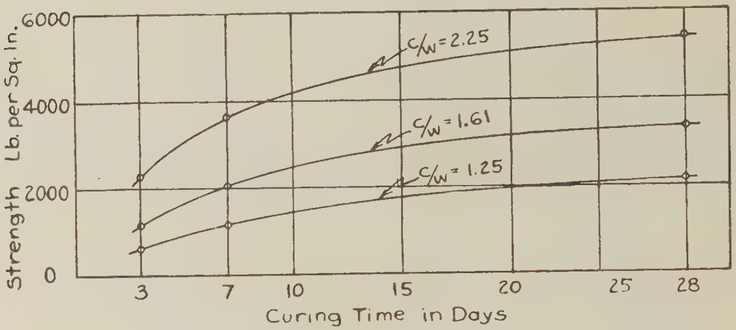


FIG. 2—AVERAGE STRENGTH—CEMENT-WATER RATIO RELATIONSHIP



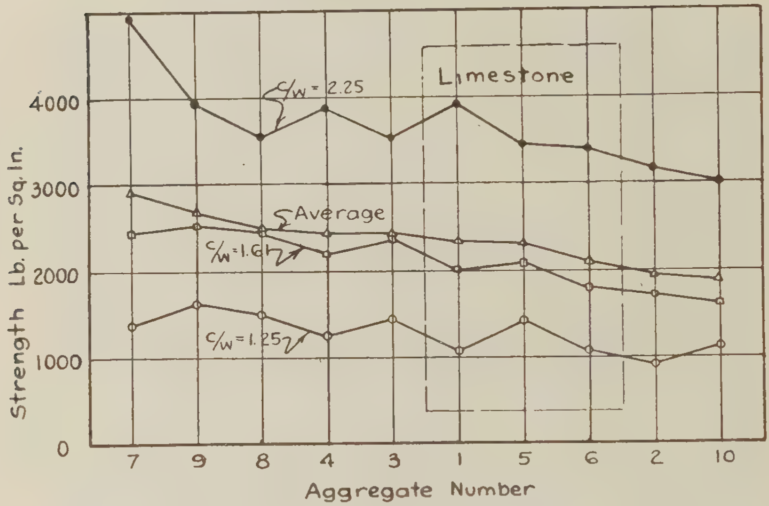


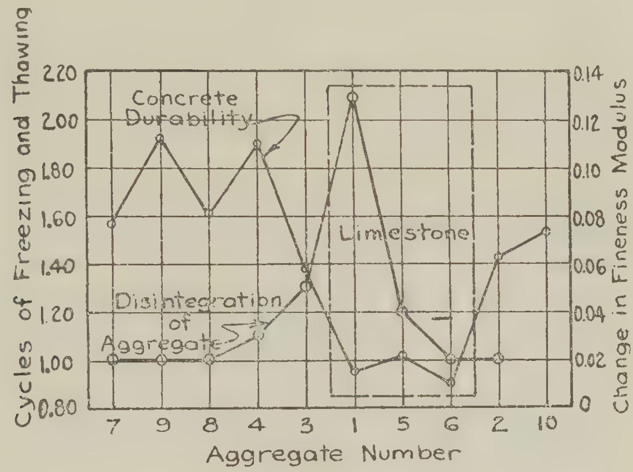
FIG. 3—EFFECT OF LENGTH OF MOIST CURING ON STRENGTH OF CONCRETE

FIG. 4—EFFECT OF TYPE OF AGGREGATE ON COMPRESSIVE STRENGTH OF CONCRETE

VI. RESULTS

The results of the tests of the aggregates are presented in Fig. 1. A fair correlation was obtained between the results of the two different tests. The change in fineness modulus was used as the basis of comparison.

Fig. 2 shows the relation between the strength and the cement-water ratio for the concrete. Each strength is the average for all the ten aggregates. The effect of the length of moist curing is shown in Fig. 3. The effect of the type of aggregate upon the strength of the concrete is shown in Fig. 4 for the three cement-water ratios used. It is noted



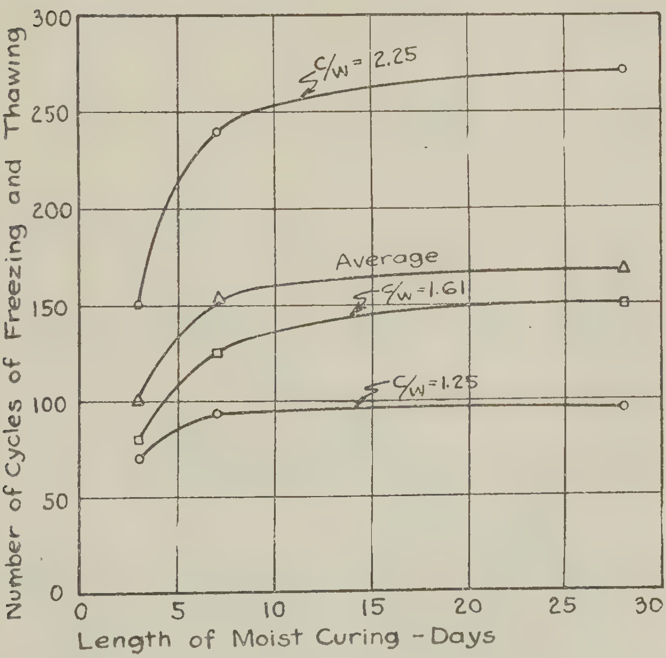
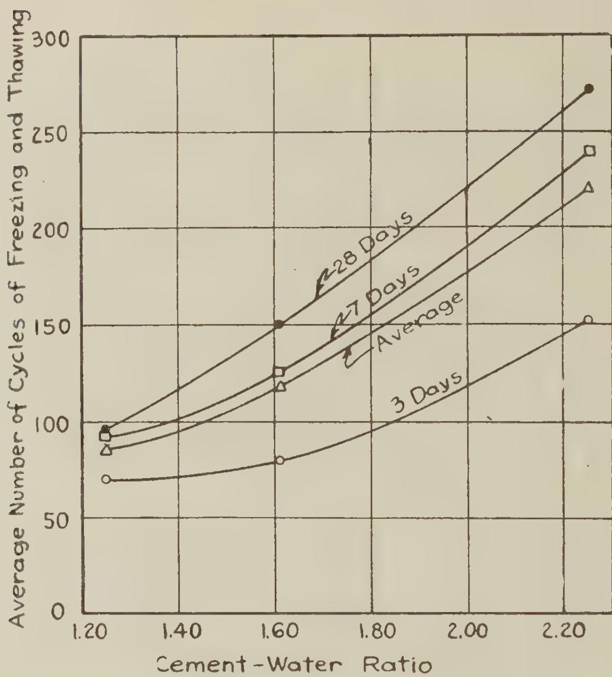


FIG. 5—CORRELATION BETWEEN DURABILITY OF CONCRETE (25 PER CENT LOSS IN WEIGHT) AND DISINTEGRATION OF AGGREGATES (AFTER 15 CYCLES OF TEST)

FIG. 6—EFFECT OF LENGTH OF MOIST CURING ON DURABILITY OF CONCRETE (25 PER CENT LOSS IN WEIGHT)



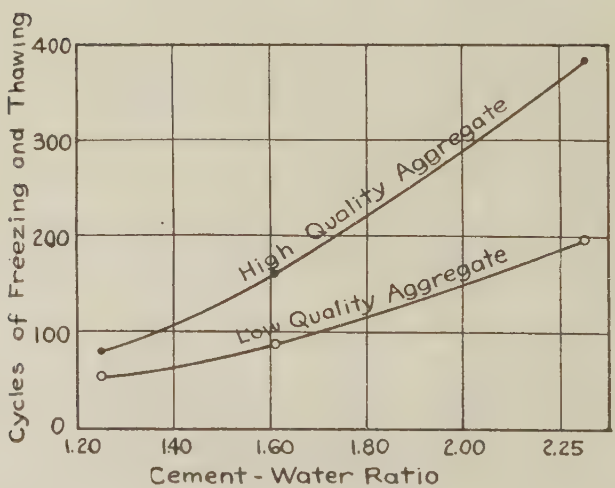


FIG. 7—EFFECT OF QUALITY OF PASTE ON DURABILITY OF CONCRETE (25 PER CENT LOSS IN WEIGHT)

FIG. 8—INTERRELATION OF QUALITY OF AGGREGATES, CEMENT-WATER RATIO OF PASTE, AND DURABILITY OF CONCRETE (25 PER CENT LOSS IN WEIGHT)

that there is a considerable variation in the strength due to the properties of the aggregates.

The sodium sulphate tests of the concrete specimens gave no indication of their durability qualities. In general, the compressive strength was increased considerably and had no bearing on the durability of the concrete.

In the freezing and thawing tests of the concrete specimens, a loss of 25 per cent of the original weight was taken as the basis for comparison. The durability of the concrete has therefore been measured by the number of cycles of freezing and thawing required to reduce the weight 25 per cent. In Fig. 5 the average results for each kind of aggregate have been plotted for the tests on concrete and on aggregates. While there is no direct relationship between the results of the two tests, those aggregates which show the largest change in fineness modulus generally give low durability of the concrete. It is noted that for the materials included in this investigation the concretes containing limestone aggregates show a degree of durability which is considerably below that of the natural sand and gravel concretes.

The effect of the length of moist curing on the durability of concrete is shown in Fig. 6. The durability increases very rapidly between the ages of 3 and 7 days, while the increase between 7 and 28 days is comparatively small. A comparison between Fig. 3 and 6 shows that the effect of early moist curing is more prominent for the durability of the concrete than for its strength.

The effect of the quality of the cement paste upon the durability of the concrete is shown in Fig. 7. The durability of the concrete increases with the increase in the cement-water ratio of the paste in much the same manner as shown in Fig. 2 for the strength. This is particularly true when the concrete has been given sufficient moist curing.

The concrete having Swedish iron ore and steel punchings for aggregate showed no corrosion during the freezing and thawing tests.

VII. DURABILITY AS AN ECONOMIC PROBLEM

A durable concrete can be produced in a number of different ways. High quality cement, a high cement-water ratio of an ordinary cement, long moist curing, high quality aggregates, or lower quality aggregates in a high quality cement paste may be used for the production of a durable concrete. The problem of the designer is to secure concrete of a given durability at the lowest cost. With the proper knowledge of the quality-giving properties and the price of the different materials, it is a simple matter to select that condition which will give the dur-

ability of the concrete at a minimum of expense. From Fig. 7 it is noted that the durability corresponding to 100 cycles of freezing and thawing may be obtained by the use of three days moist curing and a cement-water ratio of 1.85 or 28 days moist curing and a cement-water ratio of 1.30. Similarly, Fig. 8 shows that 100 cycles of durability may be obtained by the use of high quality aggregate in a 1.35 cement paste or a lower quality aggregate in a 1.70 paste. The two aggregates shown in Fig. 8 are the two extremes used in this investigation. The usefulness of an aggregate should therefore be based upon its economy in a concrete of a given quality rather than upon tests on the aggregate itself.

For such discussion of this paper as may develop readers are referred to the JOURNAL *for May-June, 1934. Discussions should be available to the Secretary by April 1, 1934.*

Earthquake Damage to Masonry Structures and Their Repair*

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INTRODUCTION

The buildings of Long Beach, California, and vicinity were given a severe test on March 10, 1933, when earthquake tremors shook the region. Losses will total approximately $200,000,000 and no building escaped without some damage. It is proposed here to pass on to the profession some of the lessons learned and to show some of the repair methods used on brick and concrete structures.

DAMAGE

Inspection of many brick structures revealed that a majority of failures occurred at corresponding locations in the walls and were namely of two types: diagonal fractures that usually followed mortar joints, and horizontal shear cracks. In some buildings there was evidence that the two parts of the brick wall had separated in the first part of the shake and then shifted laterally with respect to each other during the latter part of the shake. Fig. 1 shows a typical diagonal failure. Such damage was found at the ends of walls and at the corners of openings. The horizontal shear failures were to be found near the top of the footing and at the ceiling joist lines. Many of these cracks extended the entire length of the wall and ranged from a slight lateral movement up to several inches. Mortar used in many of the brick buildings was so soft that it could be crushed between one's fingers and it seems to have been common practice only partially to fill the joints with mortar. The Long Beach building code now requires cement mortar with full shoved joints. Fig. 2** is a good illustration of the seriousness of the damage done to some of the Long Beach city school buildings. Had the shake come in school hours, there must have been a considerable loss of life.

*Received by the Secretary, Sept. 11, 1933.
†Building Department, City of Long Beach, Calif.
‡Gunite Inspector, City of Long Beach, Calif.
**Courtesy Winstead Bros. Photograph Co., Long Beach.

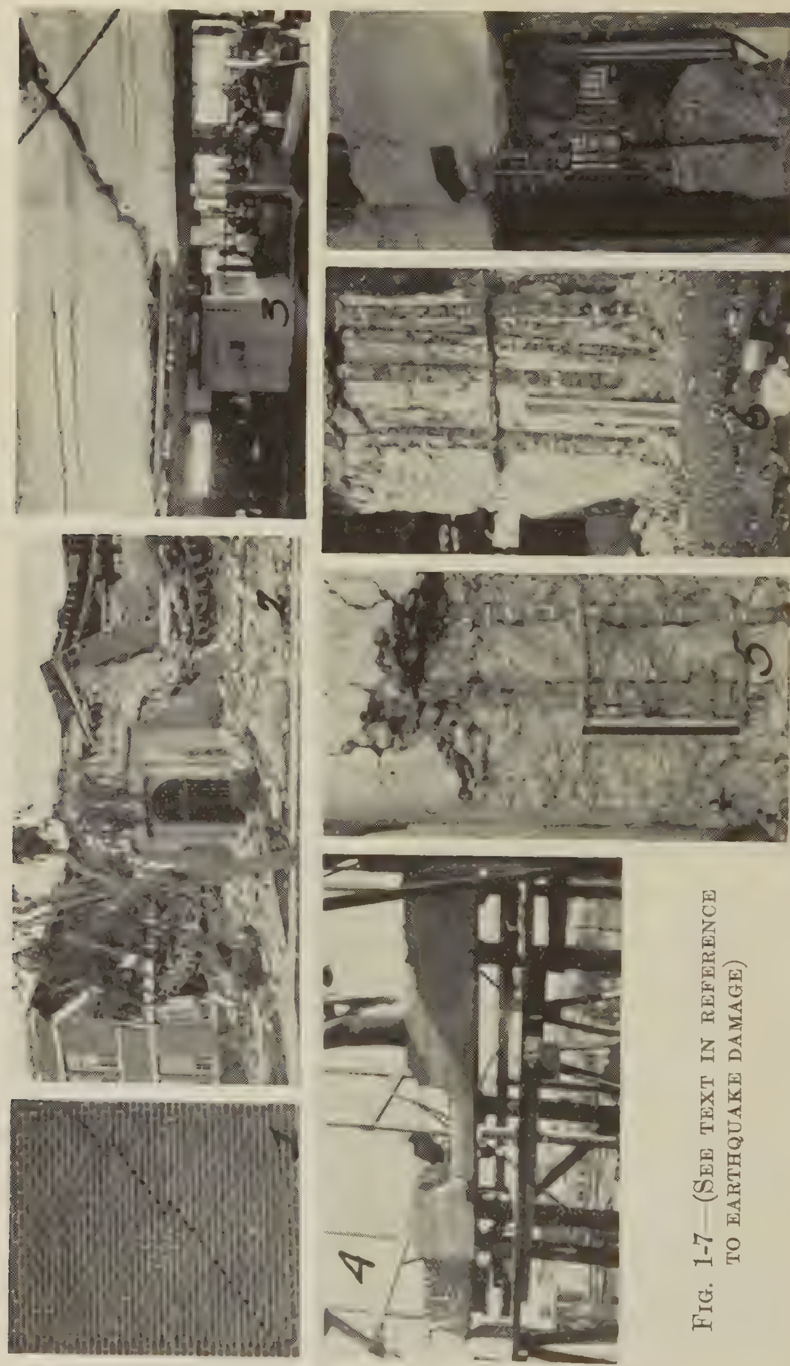

FIG. 1-7—(SEE TEXT IN REFERENCE TO EARTHQUAKE DAMAGE)

The reinforced concrete structures withstood the earthquake somewhat better than the average brick structure. Most of the damage found in such buildings was due either to poor design or poor construction. Illustrative of poor design was one building in which the designer had used deep, long span girders and very light columns. While this construction may be justified by using some coefficient of wl^2 yet an analysis of the members as a rigid frame shows the column to be overloaded by the gravity loads. It is impossible to hope that such overloaded columns would withstand additional earthquake loads. These columns showed failures at the points of high stress as determined by the analysis. The writers were pleased to note the agreement of the mathematical analysis and the actual damage. Designers overlooked the fact that most reinforced concrete structures are rigid frames. On two reinforced concrete buildings the main failures were at points of high negative moment and little reinforcing steel. There has been much neglect of the rigid frame moments in a structure while using some coefficient of moment.

While it is possible to obtain calculations that show no need of stirrups in the center of the span, yet the writers feel that good details should include some stirrups throughout the length of the beam. Fig. 3 and 4 show beams that failed because of lack of stirrups at points where they were considered unnecessary. In some members there was less than ½ in. between bars.

Important as the design is the construction. Evidences that one or two contractors were lax in their construction methods appear in Fig. 5, 6 and 7. Fig. 5 shows actual damage to a column and it is to be noted that the hoop at the lower end of the scale apparently did not act. Note the kinks in the bar. Fig. 6 shows damage to another column at the floor level. Note lack of care in spacing the bars. Fig. 7 shows still another column in the same building after removal of damaged concrete. This picture shows the position of the core relative to the center line of the column. Also note the condition of the spiral at the point of moment in the column.

METHODS OF REPAIR

Three main types of repair were used for the damaged brick structures. The brick were taken down to the lowest crack and relaid according to the new code requirements of cement mortar, full shoved joints and reinforced concrete ties; or the cracks were repaired by removing brick along each side of the crack and replacing either with brick or concrete keys; or the damage was repaired with a cement-sand mixture placed pneumatically by means of a cement gun com-

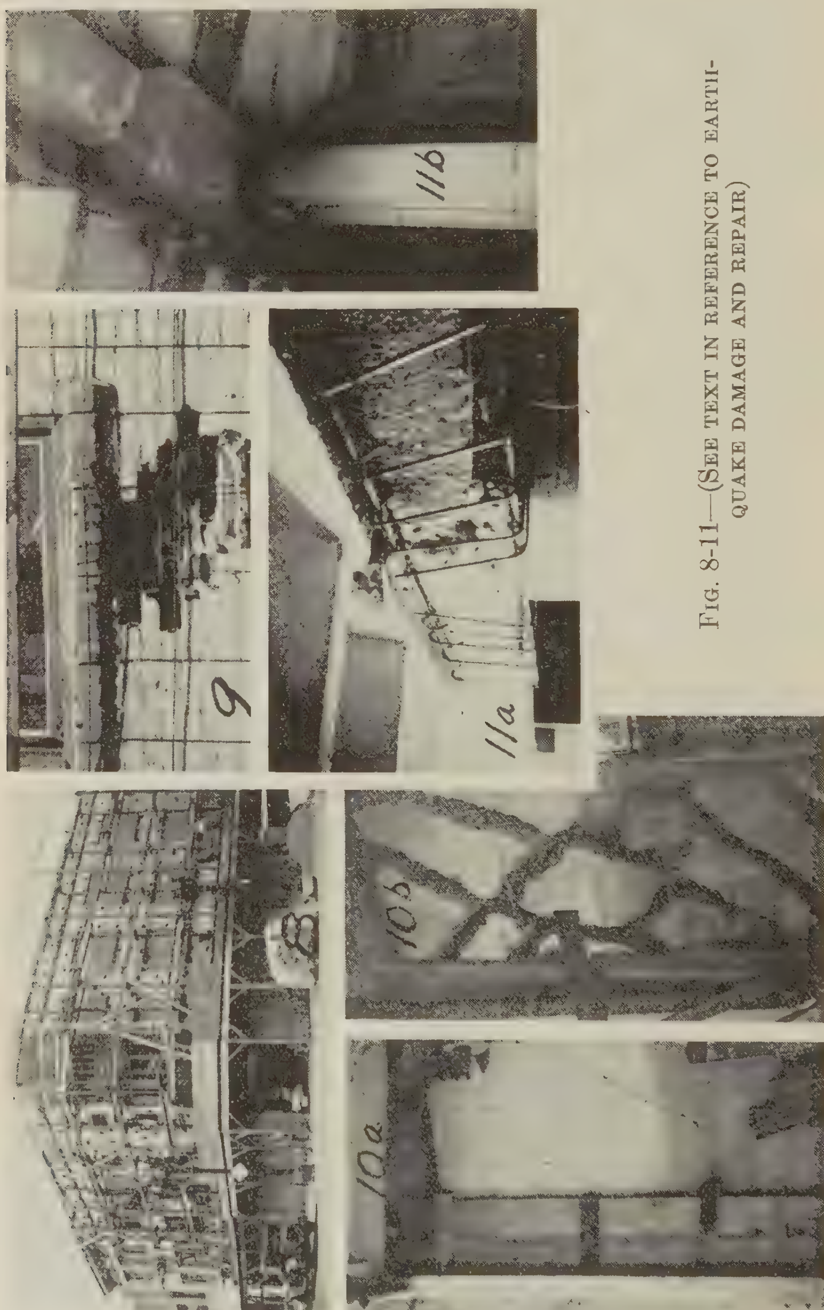

FIG. 8-11—(SEE TEXT IN REFERENCE TO EARTHQUAKE DAMAGE AND REPAIR)

monly and hereinafter called gunite. Gunite repairs may be of two main types: by chipping out the crack in a "V" notch on each side of the wall and shooting full of gunite, or the crack may be notched on the inside and the entire outside surface given a coat of gunite reinforced with mesh or light bars. Various combinations of these types of repairs have been used.

Fig. 8 gives an idea of the number of keys required on some of the brick buildings while Fig. 9 shows a close-up of a repair made up of a key and a reinforced gunite coat on the outer face of the wall.

Most repairs to reinforced concrete structures were made by means of gunite "welds." Fig. 10(a) shows a damaged reinforced concrete wall while Fig. 10(b) shows the method of repair. All damaged concrete was carefully removed and replaced with gunite. In such repairs it was necessary to remove only the damaged material and replace with new, but in the repair of beams and columns it was necessary to add new reinforcing steel. When the fracture was near the center of the beam it was required to add more steel in the bottom of the beam and sometimes stirrups, but Fig. 11(a) shows a repair at the junction of a beam with a column. The old concrete is chipped away and additional steel added as shown. The entire area is then shot with gunite and is then as shown in Fig. 11(b). Were the failure at an exterior column, then the beam bars were bent around the column and back into the beam and the entire area gunited.

Fig. 12(a), 12(b) and 12(c) show the method of reinforcing a column.

One special case of repair was so interesting that it will be included. A building had a tower made up of a reinforced concrete frame and brick filler walls and facing. The tower columns were damaged and it was necessary to transfer the load from the columns to the structure below while the repairs were made. At a point several feet above the damaged area a platform was built around the column. Reinforcing was then placed around the column as shown in Fig. 13(a). After this was in place, a gunite lug was shot on the column as shown in Fig. 13(b). As soon as this had hardened, shoring was placed under the lug and the column cut away as shown in Fig. 13(c). It was then possible to repair the damage of the column and as soon as the column section could take load, the shoring and lug were removed. Each column carried a load of 70 tons.

Another method of repair is indicated in Fig. 14 and 15. The damaged existing concrete was chipped to receive a structural steel column and truss girder. This steel was shop-fabricated as much as possible. The assembled reinforcing was then pre-stressed by means

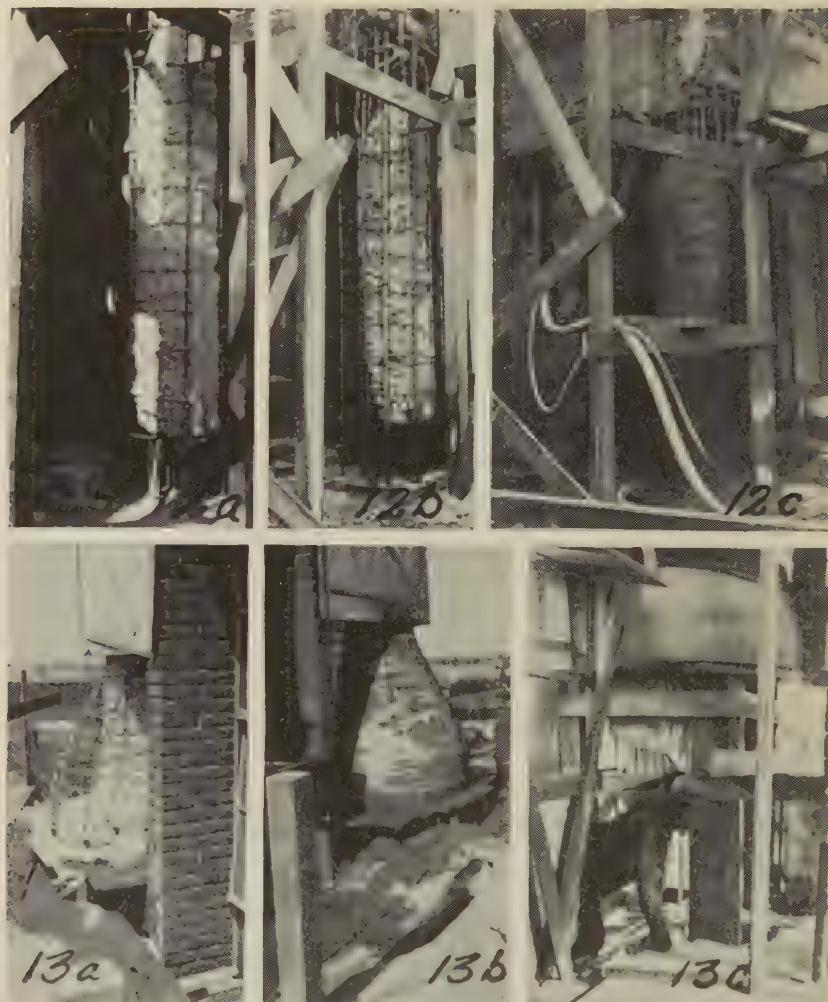

FIG. 12, 13—(SEE TEXT IN REFERENCE TO EARTHQUAKE DAMAGE AND REPAIR)

of two 35-ton jacks and welded in place. The entire section was then covered by ¼-in. fireproofing over the greatest projection.

GUNITE TESTS

Tests were made to determine the bond of 1:4½ gunite to hardened concrete. Specimens tested for shear between the gunite and old concrete showed unit shears of 380 p. s. i. (average) for 24-hour gunite and 430 p. s. i. (average) for 72-hour gunite. When beams were made

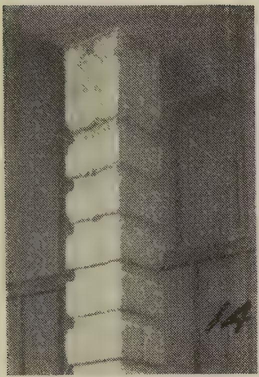

FIG. 14, 15—(SEE TEXT IN REFERENCE TO EARTHQUAKE DAMAGE AND REPAIR)

up partly of old concrete and partly of new gunite, a modulus of rupture of 315 p. s. i. (average) was developed for 24-hour gunite and 340 p. s. i. (average) for 48 hour gunite. In both cases it was the concrete that failed and not the gunite.

Practice has proven that bush hammering concrete surfaces, against which bond is to be made with gunite, is very detrimental to the bond as the concrete will be fractured for a slight depth, possibly only ⅛ in. to ¼ in., and weakened. Other methods of obtaining a bonding surface will give better results. Tests to determine the strength of a gunite weld in compression gave the following stresses: for a weld in a "V" notch, the concrete crushed at a stress of 1700 p. s. i. in the gunite, but for a weld between two square cut surfaces of concrete, the concrete crushed at a stress of 2008 p. s. i. in the gunite. Tests were also made to determine the efficiency in regained bond of new gunite to reinforcing steel. Test cylinders 6 in. in diameter by 12 in. high were made with a 1-in. diameter plain bar. When the bar was left undisturbed for seven days and then pulled, it was found that a load of 20,000 lb. was required for a slip of 0.025 in. When, however, a gunite test cylinder was seven days old and was then chipped so as to expose 60 per cent of the bar area and then re-shot with gunite and allowed to set for seven days, the load necessary to produce a slip of 0.025 in. was 18,900 lb.

Compressive tests on 6 x 12-in. gunite cylinders, of the 1:4½ mix, average 2500 p. s. i. for 24-hour gunite and 6000 p. s. i. for 7 day gunite.

REMARKS

Among the many lessons to be learned from the damaged buildings are the following:

(1) Designers should provide connections to transfer moment from one member into the other.

(2) Designers should design the entire structure as a statically indeterminate structure so as to have all members of equal strength.

(3) Detailers should give more attention to such details as the bar spacing so as to allow enough concrete to transmit the stresses from bar to bar, adequate anchorage for face brick or stone, and especially, care should be taken with stair details. One building had stair slabs with number nine wire; another had enough steel but only 4 in. of bond at the ends.

(4) The man responsible for the entire layout should avoid "unbalanced" buildings and should give considerable thought before the placing of heavy loads at the roof level, such as large water tanks or massive ornamentation.

(5) Designing engineers should have a closer and more responsible supervision of the construction.

For such discussion of this paper as may develop readers are referred to the JOURNAL *for May-June, 1934. Discussions should be available to the Secretary by April 1, 1934.*

Plastic Flow in Plain and Reinforced Concrete Arches*

BY E. PROBST†

TRANSLATED FROM THE GERMAN BY INGE LYSE

The importance of plastic deformation on the stress distribution of plain and reinforced concrete sections is properly receiving more and more consideration. It is not to be expected, however, that the final solution of different problems will be arrived at except by experimental methods. The question of how the plastic deformation may deviate from the elastic deformation due to load and time effects, the effect of shrinkage, temperature and moisture variations, can only be solved by means of extensive individual experimentations.

The cements, gradations, water-cement ratios, and methods of curing produce, as R. E. Davis has correctly stated, large variations in the plastic deformations.[1] In view of the special properties of the concrete materials it seems to me that new methods of design which take the plastic flow into consideration above all must result in a simplification. I therefore consider it out of place to try to refine present methods. I welcome the statement by Whitney[2] that for the present only average values permit a sketchy picture of the mathematical prediction of the behavior of the material.

The plastic flow of the concrete is, no doubt, a property which, in opposition to the many attempts of purely mathematically inclined investigators, permits simplification in the method of design. The effect of the equilization of stress distribution at early ages, the effect of plastic shrinkage and other factors which have beneficial effect upon the structure, do not require the development of a new theory but an expansion of the present methods in the direction of including the properties of the materials.

Whitney has used the plastic deformations only for dead load design of his arches. He assumes that the results for the live load agree closely enough with the theory of elasticity. For ordinary conditions

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[1] "Flow of concrete under the action of sustained loads" by R. E. Davis and H. E. Davis; Journal, Amer. Concrete Inst., March, 1931; *Proceedings*, Vol. 27, p. 837.

[2] "Plain and reinforced concrete arches," by Charles S. Whitney, Journal, Amer. Concrete Inst., March, 1932; *Proceedings*, Vol. 28, p. 479.

this assumption may be justified. However, for certain structures the live load may produce considerable plastic deformations. I am particularly thinking of plain and reinforced concrete arches which are subjected to very rapid repetitions of live loads. Bridges for express trains in large cities, from all stations connected with these bridges, are subjected severely to live loads. Express traffic in both directions stresses the material by relatively slow and therefore more effective repeated loading cycles. The effect on the plastic flow of the concrete is here a question of the time or the speed of loading, as well as of the intensity. Attempts have been made to decrease the great intensity, which is produced by the frequently repeated loads, by absorbing it in a large mass. An example is found in the arch structures for the city traffic system of Berlin. As, however, the time of vibration is longer for heavy structures than for light structures, and therefore at higher repetitions of loadings superimposures may occur, it is possible that a light construction may serve better than a heavy one for traffic loads.

The question of the speed of the load has not been sufficiently studied. The limitations may be of special interest here. What speed of loading produces the greatest plastic deformation? Where is the upper limit at which interference results from superimposed vibrations become apparent and threaten the safety of the structure? These questions have probably not been studied, at least not for arches of plain or reinforced concrete. Investigations into these problems can only be carried out with test methods which are superior to those now used. An observation of the crown deflection, the customary method, can give no complete answer. In addition to the deformation due to sustained load, it seems that to secure the results of vibrations and deformations due to live load it is important to take measurements at different points and in different directions, and also in the interior of the arches, that a clear picture of the stress distribution may be obtained. Perhaps electrical remote instruments, which measure both the temperature and the strain, may give the necessary information in these respects.

Now we have to be content with studying the effect of repeated loading in laboratories. The simplest form of repeated loading for the observation of the difference between elastic and plastic deformation is no doubt that of pure compressive stress. Some time ago four groups of compression prisms were tested in my Institute in Karlsruhe at the ages of 8 weeks, 10 weeks, 7½ months, and 2 years 7 months.

At the beginning of the repeated loading the ratio between the elastic and the plastic deformations was 4:1 for the 8 weeks specimens,

11:1 for the 10 weeks, and 31:1 for the 2 years 7 months specimens. The distance between the elastic and plastic deformation increased with the age of the concrete. The elastic deformation takes less part in this difference than the plastic one, because the increase in elastic deformation with age is small, while the decrease in plastic deformation is very large. In other words, the plastic flow decreases much faster than the elasticity increases with the age of the concrete. The repeated loadings changed the plastic deformation materially, but had small effect upon the elastic deformation. The increase in plastic deformation is naturally greater, the less the age of the concrete. The ratio between the plastic longitudinal deformation after the first loading and after the completion of the repeated loadings was 1:11 for the 10 weeks old specimens, and 1:1.8 for the 2 years 7 months old specimens. Both groups had more than one million repetitions of loading. Not only the dead load but also the live load had the greatest effect on concrete at early ages.

The ratio between the elastic and plastic deformations at the completion of the tests was 1:1.47 for 8 weeks old specimens, 1:1 for 10 weeks, 1:0.2 for 7½ months, and 1:0.06 for 2 years 7 months old specimens. A comparison between these values and those at the start of the repetition of loading reveals an increase in the plastic deformation, while the elastic deformations remained nearly constant and it also reveals the great effect of the age in decreasing the plastic flow.

It is of interest in this connection to study the question of what *method* of loading, dead or live load, produces the largest plastic deformations. For given material, curing and amount of load, the answer is to a large degree dependent upon the time, and therefore upon the speed of loading. When I present the following two examples from investigations in my Institute, they are for the purpose of illustrating the point. For the actual solution of the question it is necessary to carry out extensive investigations which to be applicable to the practical field must be supplemented with observations on structures in service.

For a beam of 3.0 meter (about 10 ft.) span, first subjected to repeated loadings and then placed under a sustained load, the results showed that the sustained load of 7200 kg. (15,900 lb.) in 12 days produced a greater plastic strain in the compressive section of the concrete than the previous 1,100,000 repetitions between 600 kg. (1320 lb.) and 7200 kg. A rate of 90 repetitions per minute was used. Another beam of the same size and shape was first subjected to a sustained load of 7200 kg. for 23 days, and then given 242,000 repetitions

at a rate of 90 per minute between loads of 600 kg. and 7200 kg. These repetitions did not increase the deformation in the compressive section of the beam beyond what was produced by the sustained load. When the rate of loading was reduced from 90 to 22 per minute, a reduction of about 75 per cent in the rate, the plastic deformations increased, while the elastic deformations remained stationary. The plastic deformations reached a maximum at 272,000 cycles, which was not exceeded by an additional 30,000 cycles. As the original strength of the concrete had not been exceeded and the beam was tested at a relatively great age, the decrease in the plastic longitudinal deformation is natural. The total deformations in the compressive section of the first beam were approximately equal to those of the second beam, although the succession of loading was reversed in the two beams.

The example illustrates well the importance of a change in the speed of loading. Uniform speed has less effect on the concrete than a variation in speeds. The irregularity in the speed of application of live loads may stress the material to a higher degree than a regularly repeated loading.

I may also present an example of how the plastic deformation in the compressive section of a beam subjected to high frequency repetition affects the reinforcement in the tensile section. A beam of 3.50 meter (about 11½ ft.) span was subjected to approximatey 650,000 repetitions of between 1000 and 15,100 kg. (2200 and 33,000 lb.) at a rate of 90 per minute, at the age of 10 months. At first it was loaded gradually to the upper limit of the repeated loading, the longitudinal deformations in the compressive and tensile sections, and in the reinforcement, increasing in direct proportion to the increase in load as long as no crack developed. The first crack was observed at a load of 4800 kg. (10,600 lb.). The original line for the total deformation showed a curvature at the development of the first crack. The longitudinal deformation in the steel increased rapidly, and so it did in the compressive section at the top of the beam, while at 9 cm. (3½ in.) from the top of the beam, the rate of increase in the compression deformation decreased, indicating an upward movement of the neutral axis.

Before the application of the repeated loadings the deformation curves for the compressive section and for the reinforcement were nearly straight lines. The repeated loadings produce the same type of deformations as those produced in direct compression tests, namely an increase in longitudinal deformations. The elastic deformation reached its stationary value before the plastic deformation. Due to the formation of cracks and the plastic flow of the compressive section,

the stress in the longitudinal reinforcement increased with the repetition of loading. For the first application of 15,100 kg. the strain in the reinforcement was 548 millionths (stress 16,500 p. s. i.) and reached 694 millionths (stress 20,800 p. s. i.) at the limiting value. Upon unloading to 1,000 lb. a strain of only 468 millionths (stress 14,000 p. s. i.) was measured, although the steel had been stressed within the elastic limit. The remaining strain of 226 millionths (stress 6800 p. s. i.) becomes a permanent strain in the steel. Although the steel had been stressed to less than its elastic limit a permanent stress was produced in the reinforcement by the plastic deformation of the compressive section. The tensile portion of the concrete remained unaffected. Measurements between two cracks in height with the reinforcement gave practically no deformation in the tensile strain of the concrete. The effect of the permanent stress in the steel on the cracks seems to remain as long as the deformation is within the elastic limit. The cracks open and close almost completely. Their expansion was stopped, as far as could be observed, at 387,000 repetitions. No doubt this permanent stress in the steel is able to prevent a further plastic flow in the compressive section of the concrete as long as the repeated load is applied and the concrete is subjected to a sustained load.

If practical application is made of these laboratory results, it appears evident that more attention should be given to the reinforcement in the tensile section. The plastic flow of the compressed concrete produces an effect in the steel which in turn prevents further plastic flow.

When we consider the favorable effect of the early completion, we should also remember that an arch with heavy reinforcement partly offsets these advantages as the stresses in the steel may exceed those intended by the designer. I would in such cases recommend that to prevent overstressing of the steel from the plastic flow of the fresh concrete, the live load be not applied too early to the reinforced concrete arches. The later the application of the live load, the more will the reinforced concrete arch approach the behavior of a true elastic arch under repeated loadings.

For such discussion of this paper as may develop readers are referred to the Journal *for May-June, 1934. Discussions should be available to the Secretary by April 1, 1934.*

Selecting Concrete Plant—Meadowbrook Hospital*

BY JOHN G. AHLERS†

MEMBER AMERICAN CONCRETE INSTITUTE

The analysis of five types of plant for the construction of Meadowbrook Hospital illustrates a method of arriving at a solution of the problem of obtaining the best and most economical plant layout.

The work consisted in constructing foundations and concrete framework for six buildings and 920-lin. ft. of tunnels at the Nassau County General Hospital, Hempstead, Long Island. The work was done by Barney-Ahlers Construction Corp. as subcontractors for C. T. Wills, Inc.; the office of John Russell Pope, architects. Work was started July 1,and concrete framework completed December 1, 1932.

Though requirements for the quality of concrete were rigid, the specifications were written with appreciation of the difficulties of producing good concrete; provided incentive to careful plant installation and allowed the contractors to profit by a knowledge of proportioning and mixing concrete.

The general layout gives an idea of the problems involved in deciding on the best plant. Some immediate questions were:

A—Were individual plants for each structure preferable to a central plant or to buying a ready mixed concrete?

B—If a central plant were erected on the site, where should it be located?

C—If a central plant, how should the concrete be distributed—

1—wet mixed, or 2—dry mixed in transit, or

3—by chutes, or 4—by conveyors?

D—If a central plant, should this be made portable and moved from building to building?

Ready mixed concrete from any nearby plants proved so high in price that a plant on the job could be definitely decided upon. Calculation of rental or cost of transit mix trucks eliminated this type from consideration, though for a somewhat larger job this type of plant

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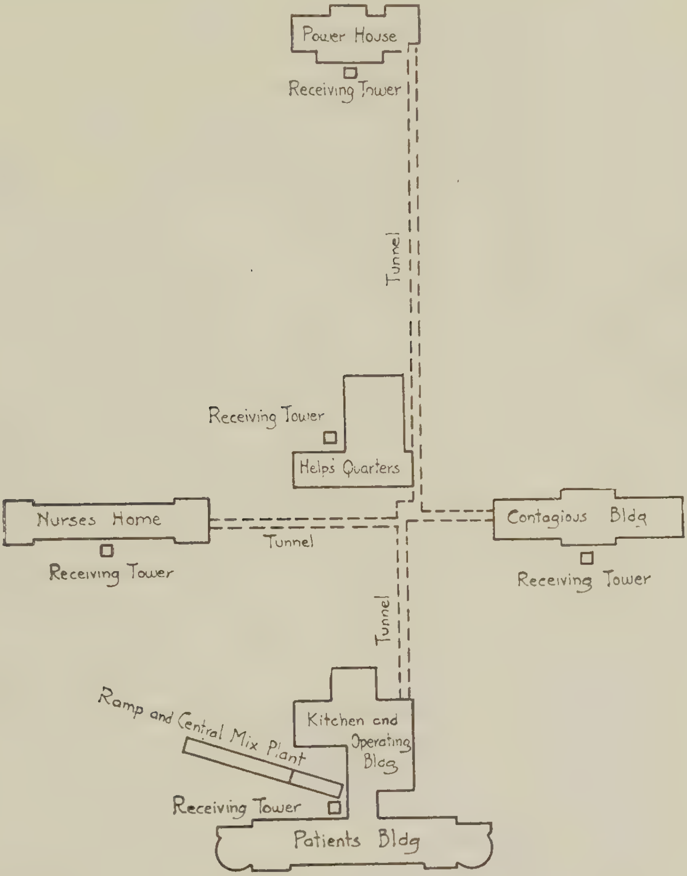

FIG. 1—PLOT PLAN FOR MEADOWBROOK HOSPITAL

could be considered. Bad road conditions were an important consideration; transit mix trucks weighing at least ten tons would be mired in rainy weather. A dry batching plant was thus eliminated from consideration.

To arrive at a conclusion certain costs had to be fixed, and the rates in Table 1 were set as fair on which to base comparative estimates.

The study reduced itself to consideration of five possible plants; three individuals in the contractor's organization were assigned the task of making independent estimates in the form shown in Table 1, but with all figures omitted except the units costs given.

TABLE 1—RATE BASIS FOR ESTIMATES

	Cost	Rental
1 yd. mixer	——	$ 65.00 wk.
1 yd. hoist	——	65.00 wk.
1 yd. bucket	——	7.50 wk.
Steel headblock	$100.00	——
½ yd. mixer	——	35.00 wk.
½ yd. hoist	——	46.00 wk.
½ yd. bucket	——	4.00 wk.
Headblock (wood)	75.00	——
Trucks (Ford)	500.00	25.00 wk.
Wood tower	1.00 per ft.	——
Wood tower erection	2.00 per ft.	——
¾ yd. mixer	——	45.00 wk.
¾ yd. hoist	——	46.00 wk.
¾ yd. bucket	——	6.00 wk.
Paving mixer with tower 65 ft. high	12,000.00	500.00 month
Platforms	50.00	——
Floor hoppers	15.00	——

While the three estimates varied in their totals, they showed the same percentage of difference between types. Two estimates showed a variation of less than one per cent in totals.

Table 2 is self-explanatory and shows the detailed breakdown for the main building only—a similar detailed cost analysis was made for each of the other structures but here only summarized.

The final costs were sufficiently close so that any factors not contained in these estimates had to be weighed. The essential points considered were the following:

A—Speed—Work should be finished before cold weather to prevent heavy cost of protection. Plants 1 and 2 guaranteed rapid progress, 3 and 4 would be slower, particularly 4 which would permit erecting but one building at a time. Cost of forms entered here and suggested a schedule in which three buildings were under way at one time—this favored plant No. 2.

B—Road conditions—Were of the worst—the job in the middle of a farm with a two foot top soil and very poor drainage; this favored plant No. 5. Tests on trucks had shown that plant No. 2 was better than No. 1, as light Ford trucks with four rear wheels could travel over fields in any condition.

C—Concrete control for quality and consistency—Here the balance was in favor of plant No. 2 and this was an important factor in the final selection.

D—Number of trucks required—There was considerable diversity of opinion on this point; it was decided to try two and buy or rent more if required. (As it later turned out these two, at a speed of from 20 to 40 miles per hour drove all over the site and there never was any delay waiting for the transportation of the concrete except for an occasional flat tire or broken spring. Any delay was invariably in disposing of the concrete at the point of delivery.)

TABLE 2—PLANT COST OF CONCRETE DELIVERED ON FLOOR IN BUGGIES (NO FINISH)—FIVE ALTERNATES FOR MEADOWBROOK HOSPITAL

ITEMS		1 Individual Plants	2 Central Plants	3 Chuting	4 Tower on Paving Mixer	5 Belt Conveyors
1. Patients bldg. and kitchen:		26 Wks.	26 Wks.	26 Wks.		
Mixer	$0.65	1040	1690	1690 –		1690
Hoist	.65	1040	1690	1690 –	3000	1690
Bucket	7½	120	195	195 –		195
Headblock		75	75	75 –		75
Tower: material 90 ft.	1	90	100	400	75	100
labor	2	180	200	600	100	200
Lumber and Miscl.		25	50	100		50
Additional sheathing			50	50		50
Ramp and bins			1000	1000		1000
Receiving sand, gravel, cement		550	0	0	370	0
Control system		100	125	125	125	125
Floor hoppers		50	50	100	50	100
Extra labor on mixing (15¢ per yard) 3700 yds.		550	0	0	550	0
2. Contagious building: 750 yards 10S mixer—same items as above—total		1228	520	440	move 445	move 600
3. Nurses home: 1160 yards ½ yard mixer—same items as above—total		1869	745	745	move 690 frame	½ cost 1000 500
4. Helps quarters: 1130 yards ½ yard mixer—same items as above—total		1638	708	708	move 683 frame	½ cost 1000 500
5. Power house: 475 yards 10S mixer—same items as above—total		1033	408	408	move 495	408
6. Tunnels:					move	
720 yards 7S mixer		300	0	0	250	0
Receiving mate.ials		72	0	0	72	0
Extra laboı on mixing		108	0	0	108	0
Control system		25	0	0	25	0
7. Waste material: 4200 yards		420	0	0	420	150
8. Loss of cement .15 bbl. per yd.		1000	0	0	1000	0
Loss cement bags 4200 bbls.		420	0	0	420	0
9. Rental or cost:						
Concrete dist. trucks		0	500	500	0	200
Drivers 4200 trips 1000 hrs.		0	600	600	0	250
Repairs and tires		0	200	200	0	75
(Maintenance 1000 hrs.) (Gas and oil)		0	333	333	0	50
10. Additional hoisting engineers, etc.		0	500	500	0	0
11. Additional cost batch delivery		0	0	0	2000	0
12. Additional labor placing 4,200 yds.		210	0	0	420	210
13. Chutes, cables and liners		0	0	1000	0	250
TOTAL		12303	9739	11459	11298	10468

E—The main plant size—A one-yard mixer was questioned as too large. (The capacity of this mixer was never taxed, but it did give an opportunity to mix the concrete from 1½ min. to 2½ min., which may account for some of the high strengths and uniformity ın the concrete. Experience has shown an over-size mixer always helps toward speed and insures capacity.)

F—Specification requirements—The architects, John Russell Pope, had written a specification that encouraged the engineers of a contractor's organization to use their brains—a specification that could well be used by other architects and is shown in part to illustrate the value of a specification that while rigid, allows opportunity for benefits from its application. The contract specification on concrete provided:

"16. *Materials and Proportions:*

(a) Cement, Aggregates, and Water shall be as specified under Concrete and Masonry Materials:

"(b) Proportions:

"All reinforced concrete except where otherwise specified or designated on the structural design drawings, shall be mixed in the proportion of one part of cement and six parts of fine and coarse aggregate, the fine and coarse aggregate to be measured separately; and not over 7½ gallons of water is to be used per bag of cement. This maximum quantity of water includes the water in the aggregate and the actual quantity to be used shall be reduced by this factor. In determining the quantity of fine aggregate due allowance shall be made for "Bulking" due to the presence of moisture.

"The actual proportion of fine to coarse aggregate shall be determined on the job, subject to the approval of the Architect and the proportion of fine aggregate shall be between the limits of 40% to 60% of the coarse aggregate when measured dry.

"Where specified on the drawings as 1:1½:3. the proportions shall be one part of cement to four and one-half parts of fine and coarse aggregate and not over 6¾ gallons of water per bag of cement shall be used. The same regulations in the use of this proportion shall apply as for the 1:6 concrete.

"Where in the opinion of the Contractor it is necessary to use concrete of a more fluid consistency than given above, this greater fluidity shall be secured by decreasing the quantity of aggregates per bag of cement. The water shall under no conditions be increased over that specified above."

With plant No. 2, 3 and 5 a careful weighing plant could be installed that could maintain exact proportions and water.

The foregoing considerations, in addition to the costs shown in Table 2, led to the selection of plant No. 2. Plant No. 5 has many merits and would have been seriously considered but had to be ignored as no electrical power was available. A gasoline operated plant was used throughout, there were no breakdowns at any time.

The plan and section show the installation of the weighing hoppers, control apparatus, ramps and bins. The main tower was built to its full height at once and the hopper for truck drawn concrete installed on the side of the ramp.

The selection of the light Ford trucks for handling the concrete to the scattered operations proved a very happy one. These trucks traveled like little tanks anywhere and were never stuck under the most adverse conditions. These light vehicles backed up to the edge of excavations so that concrete could be chuted to most of the footings and basement walls. Only once on a false signal did one truck

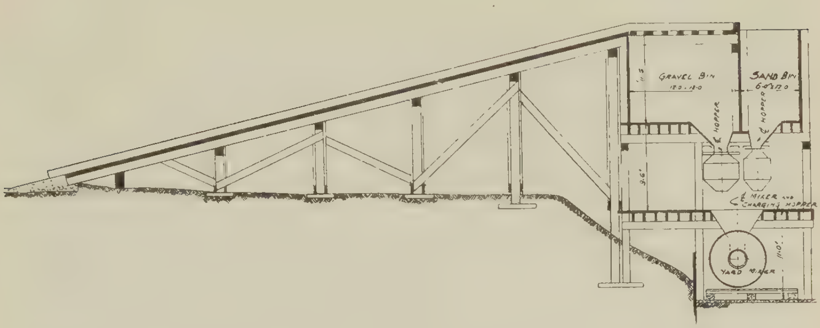

FIG. 2—PLANT NUMBER 2—AS USED ON THE JOB

get too near the edge and tumbled in, doing no serious damage, the driver jumping. The truck was pulled out and running again in an hour. The speed of the trucks over rough ground was remarkable. In fact it became somewhat of a race between drivers and a game not to delay the delivery of concrete. Even to the distant power house there was no need for more than two trucks. Maximum yardage hauled was 154 in 4½ hours to the Nurses Home.

The general plant results were fully up to expectations. Concrete was kept at a uniform strength. The table of average results of specimens indicates the results for various ages. The averages for months shows a regression for the colder periods. Cylinders were all stored in wet sand next to the job laboratory.

PLANT OPERATION—CONCRETE CONTROL

The operation of the plant was along established lines. The sand and gravel were dumped into overhead bins from which, in weighing hoppers, they were dumped into a 1-yard mixer—the water being added through a strength regulator installed on the hopper floor. Each batch of water, less the moisture in aggregate, was automatically weighed off against the required number of bags of cement.

The architects allowed a testing laboratory to be established on the job with a 200,000-lb. hydraulic machine. In this laboratory all specimens, except a set of check cylinders sent to Polytechnic Institute in Brooklyn were tested.

A very satisfactory control of aggregates was maintained by the weighing of each batch of sand, gravel, water and cement. A daily test was made for moisture and weight per cubic foot of fine and coarse aggregate. To have a plastic and workable concrete full advantage

TABLE 3—CONCRETE CYLINDER TESTS RESUME

	Average Strength p. s. i.	
	7-Days	28-Days
Patients building		
Footings	1896	2562
Walls	1815	3275
Slabs	1520	2289
Interior columns	2024	2892
Kitchen and operating building		
Footings	1840	2612
Walls	1855	2077
Slabs	1717	2556
Interior columns	1837	2804
Nurses home		
Footings	1618	2302
Walls	2107	—
Slabs	1680	2506
Interior columns	2266	2854
Contagious building		
Footings	1558	2453
Walls	1908	—
Slabs	1587	2113
Interior columns	—	—
Helps quarters		
Footings	—	—
Walls	—	—
Slabs	2377	—
Interior columns	—	—
Power house		
Footings	2180	2426
Walls	1650	2262
Slabs	1688	2381
Interior columns	—	—
Tunnel walls	1720	2185
All footings	1801	2479
All walls	1830	2766
All slabs	1661	2448
All interior columns	2038	2847
All footings plus walls plus slabs (average)	1737	2492
Average by months:		
July	1782	—
August	1755	2422
September	1771	2651
October	1449	2210
November	—	2021

was taken of the latitude in the specification and 60 per cent of fine was used. This gave an over-sanded mix but it was actually more economical to use such proportions.

A typical works test on aggregates is as follows:

DATE	SAND	GRAVEL
August 19, 1932	F.M. 3.22, Wt. 114 lb. Moisture 4%	F.M. 6.75, Wt. 112 lb. Moisture 1⅓%

Normal daily procedure was about as follows: Slabs 1:6 concrete 5 bag batch allowed 30 cubic ft. aggregate.

Gravel	X	
Sand	.60X	
1.60X = 30 cu. ft.	X =	18.75 cu. ft. gravel
	.60X =	11.25 cu. ft. sand
Gravel 18.75 at 112 lb. cu. ft. =		2100 lb.
Moisture per cent 1½ =		31.5 lb.
Setting gravel weighing hopper		2131.5
Sand 11.25 at 114 lb. cu. ft. =		1282.5 lb.
Moisture per cent 4 =		51.3 lb.
Setting sand weighing hopper		1333.8
Setting concrete strength regulator:		
7½ x 5 gals. =		312.5 lb.
Less water in aggregates =		82.8 lb.
Net Water—Maximum		229.7

To arrive at the yields of this mix, a procedure suggested by C. A. Pearson was followed, as follows:

Cement, absolute weight 5 x 94 =	470 lb.
Gravel	2100
Sand	1282.5
Water	312.5
Total	4165.0 lb.
Weight cubic foot	149 lb.
Yield cu. ft.	27.95 cu. ft. = 1.03 yd.

Average factor of fine per cubic yard of concrete = .55
Average factor of coarse per cubic yard of concrete = .67

These computations checked with the observed results when allowance was made for loss and spilling. The strengths obtained were satisfactory both to the architects and engineers, and very satisfactory for early removal of forms.

By having delivery made at one point it was possible to check every load and have assurance as well of the grading and quality of the sand and gravel. This factor alone, though it cannot be computed, may well go a long way toward paying for a central plant.

The time factor in mixing and placing received a severe test. The concrete, whenever delivery slowed up, was kept in the mixer until a truck came back for another load. There was practically no segregation due to the thorough mixing, only in isolated instances did the hoppers require prodding and ramming to release the concrete.

The concrete had a very good uniform appearance and showed up well after removal of forms. Some of the concrete work had to be ex-

FIG. 3—MEADOWBROOK HOSPITAL UNDER CONSTRUCTION—JOHN RUSSELL POPE, ARCHITECT

posed after completion and such surfaces were rubbed immediately after removal of forms, following the American Concrete Institute recommended practice.

Cost of mixing and placing concrete was low considering quality of the work and operating conditions—averaging about $1.01 per cu. yd. The cost of plant checked closely with the estimate or about $1.19 per yard, making total of $2.20 per yard for plant and labor.

The form work followed conventional lines excepting the pan construction. The problem of metal pans vs. wood cores was carefully studied. On previous contracts with pans there had been difficulty in maintaining uniform slab thickness; seepage through pans had been considerable; working conditions not good.

A type of wood cores was worked out and found satisfactory. The architects again cooperated by permitting a layout of bridging and ribs for a multiplicity of use of these cores. A complete layout was prepared, and by key numbers and letters the various types identified

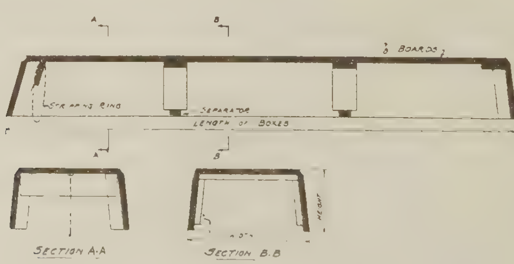

FIG. 4—TYPICAL WOOD PAN OR CORE

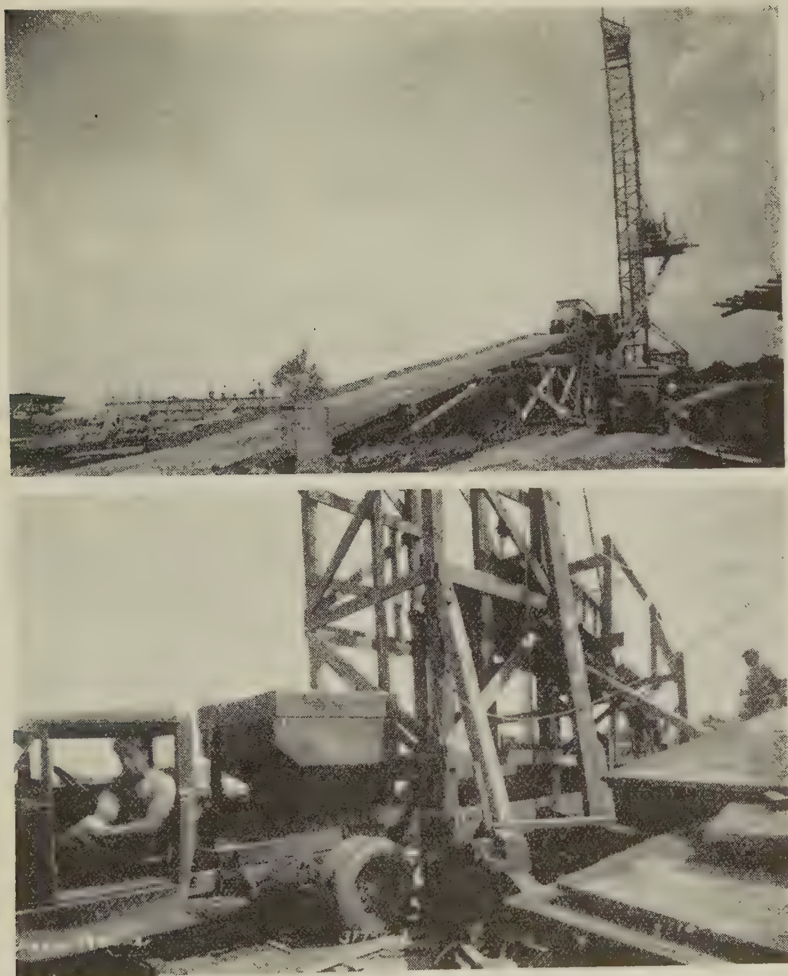

FIG. 5, 6—THE PLANT

and located. Two engineers handled this entirely with common laborers carrying the cores, and even if costs were not considerably lower the saving in materials and better working conditions fully justified their use.

Forms were removed very quickly. Standard practice was to break cylinders from any pour the same morning that the forms were to be removed and in no case to strip any horizontal member unless the

concrete showed a strength of at least 650 p. s. i. In many instances concrete showed a strength of 1100-1200 lb. at 48-hours.

CONCLUSIONS

A central mixing plant is desirable where control and quality of concrete is desirable and the specifications permit a contractor to benefit from control methods.

A contractor can, in specific instances, especially where road conditions are bad, do better with his own mixing plant than in buying concrete ready mixed from outside.

A central mixing plant gives a contractor immediate control of his operation for early removal of forms.

Capital investment for small trucks is low and reduces the cost of placing concrete in a spread-out operation.

For such discussion of this paper as may develop readers are referred to the JOURNAL *for May-June, 1934. Discussions should be available to the Secretary by April 1, 1934.*

Discussion of Report of Committee 105:

"Reinforced Concrete Column Investigation"

closure by chairman, committee 105*

F. E. Richart (*Urbana, Ill.*): The division of opinion in Committee 105, mentioned by Mr. Logeman, is not on the results of the experimental program. I believe we are in good agreement as to the meaning of the test results. The difference is very largely a matter of personal viewpoint as to the proper emphasis to be given different features of column design, with two definite schools of thought represented. The minority report follows to some degree the concept of column action generally held in Europe. As an example of European design, the regulations of the German Committee on Reinforced Concrete recognize the spiral reinforcement as 2½ times as effective as vertical reinforcement and permit spiral percentages up to three times that for the vertical reinforcement, for which the upper limit is 8 per cent of the column core area. The minority report, while not approaching the German regulations in its emphasis on the spiral reinforcement, does permit as much as 2/7 of the column strength to be furnished by the spiral reinforcement, using a maximum of 2 per cent of spirals, assumed twice as effective as vertical reinforcement. The majority of Committee 105 do not favor the utilization of a large amount of column strength produced by the spiral reinforcement when the development of this strength can begin only after the yield point of the column has been exceeded and will therefore be accompanied without exception by a large amount of inelastic shortening and bulging of the column. They prefer instead a design formula with limitations producing a column in which the deformations will be nearly elastic clear to the ultimate strength of the column. In such a column only enough spiral reinforcement is used to hold the vertical bars in alignment (even though they reach the yield point), to provide insurance against sudden failure, and to provide toughness and ability to give warning in case of overloading.

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*See discussion Journal, Amer. Concrete Inst., Sept.-Oct. 1933; *Proceedings* this volume p. 78.

Another major difference between the two reports is with regard to factors of safety, as based on yield point of column. The majority value is a maximum with low percentages of vertical reinforcement, where overstress due to plastic flow is to be encountered, and decreases as this percentage increases. In the minority report the lowest factors of safety are found with the smaller percentages of vertical steel, and the factor increases in general with the percentage.

The statement that the spiral contributes little or nothing to the column strength at working loads is correct. However, this does not mean that its contribution at the ultimate load, through the increased strength its restraint produces in the concrete, should be ignored entirely. We prefer to consider the spiral as providing a reserve of strength and toughness in the column.

The tests have demonstrated very clearly that the vertical steel can be depended upon to develop and maintain only its yield point stress, even though the final shortening of the column may be 10 or 15 times the yield point deformation. This is well shown in Professor Lyse's recent report on the tests of Series 4. In a few cases we have found hard grade steels in which tests of coupons showed that the stress increased rather quickly with increase in strain beyond the yield point. These steels produced some excess in the resulting column strength above the general rule representing the great majority of the tests.

There is no reason why we cannot calculate the strength of a column containing vertical bars of two distinct grades, if they are symmetrically placed. Each could be expected to contribute its yield point stress to make up the ultimate column load. To be sure, they might have equal stresses at working loads, but it should be evident by this time that the stresses at working loads have but little bearing upon ultimate loads or upon the permissible loads to be used in design.

From Mr. Wheeler's discussion it appears that the results from our proposed formula for spirally reinforced columns are near agreement with his recommended designs when about 1½ per cent of vertical reinforcement is used (See Tabulation A). His principal criticism appears to be in the proposed use of 8 per cent of vertical reinforcement, where he apparently would limit the percentage (on gross area) to about 2 per cent. It should be recognized that the columns listed in Tabulation B are not at all comparable, except as to diameter. The comparison ignores the amount and quality of vertical and spiral reinforcement. Conservatism in accepting the use of high percentages of vertical steel is commendable, particularly in view of the present practice in producing lapped splices on most jobs. However, the

Committee is proposing an upper limit of 8 per cent of reinforcement with the knowledge that such an amount of steel cannot be properly placed if lap splices are used, and that welded splices will be the only solution. With welded splices in use, we believe that much of the opposition to high percentages of steel will disappear. The saving in lapped bars effected through welded butt-splices gives promise of economy as well as desirable structural features in such connections.

Mr. Mensch raises a number of interesting and important points. He selects an extreme set of conditions, including a small square column with a minimum 2-in. covering outside the spiral, strong concrete and low-strength spirals, to show that special percentages as great as 4.3 per cent may be required by the proposed formulas. The reply to this is that if anyone wants to use such a column, any smaller amount of spiral reinforcement will be ineffective. The Committee recognized that its proposal will discourage the use of spirals in small columns, particularly square ones. If the spiral reinforcement costs more than one-fifth of the total cost of column materials, it would be cheaper to use a tied column at the reduced permissible stress. I wonder if Mr. Mensch recognizes that the minority formula discourages the use of practically *all* spirally reinforced columns.

The criticism of a factor of safety less than 3, when the column may be subject to bending stresses not considered by the designer is very much to the point, and the Committee will welcome the opinion of other engineers on this matter. Obviously it is much easier to select a more conservative factor of safety, but the Committee recognized its responsibility to provide for economical, as well as safe, design. It also strove to establish proper relative values of the factor of safety for tied and spiral columns. If any increase were to be made in the values that have been porposed, it should be made for both types of column. It may be pointed out that the only cases in which the factor of safety is much below 3 are for spiral columns with large percentages of longitudinal steel, which should be effective in resisting bending stresses.

Mr. Mensch attempts to show a difference in action between spirally reinforced columns with and without protective shells, by reference to the Fourth Progress Report on tests. Unfortunately, the curves are not shown completely near the ultimate load; furthermore, in the 12 and 16 and some of the 20-in. columns with shells, the strength of shell was greater than that added by the spiral, and the latter served only to produce a long drawn-out failure. Mr. Mensch's comment that while hooped columns with shells may have a large factor of safety against complete failure, but a much smaller factor against cracking or spalling of the shell is a very good argument for the type of column design

which the Committee proposes. In such a column there can be no tendency for the failure of the shell unless a load very nearly the ultimate strength of the column is applied.

It is difficult, however, to discover the logic upon which Mr. Mensch's proposed column formula is based. His equation, recognizing simultaneously both the strength of the concrete shell and that produced by the spiral reinforcement, cannot be justified by any column tests on record. As to his criticism of the limit of 8 per cent of reinforcement as too low, a similar criticism as to permissible values of spiral stress, and statements regarding the action of vertical steel with a yield point of 96,000 p. s. i., it may be said that these are controversial points which lie outside the field covered by the design formulas proposed by the Committee.

In conclusion, Committee 105 would again call attention to the fact that it is a committee on tests and not on design or specifications. Because of the long study that was made in planning the tests and in interpreting the test results, the Committee felt obligated to translate its findings into the recommended design sections that have been presented in this final report. The Institute has not been asked to adopt the report as a standard. The report must stand on its merit alone. Any adoption of this material as a standard will necessarily come through the standing committee, 501—Standard Building Code, or a similar committee on design or specifications.

Discussion of a paper by Albin L. Gemeny and C. B. McCullough:

"The Freyssinet Method of Arch Construction Applied to the Rogue River Bridge in Oregon"*

AUTHOR'S CLOSURE

Albin L. Gemeny, (Washington, D. C.)—The authors of this paper were pleased to have the valuable comments of an engineer of Mr. Whitney's achievements in the field of concrete engineering. We hope that he will find the time to make a detailed study of the complete data which are presented in the final report on the project.†

It is presumed that Mr. Whitney's disagreement with the conclusions as to the efficacy of this method in eliminating rib shortening stresses and attaining greater economy in arch construction is based upon the contention that rib shortening stresses are relieved, at least partly, by plastic flow of the concrete, and that in the design provision need be made for only a fraction of the stresses which would occur if concrete were a perfectly elastic material. We thoroughly agree that plastic flow does relieve such stresses but we do not believe that our present knowledge of the relation between shrinkage and plastic flow of concrete permits us to evaluate such effects, even roughly, except under completely controlled laboratory conditions. Unfortunately, the conditions under which arches are actually built, the concrete proportions generally used and the moisture conditions which usually exist tend to reduce greatly this flow in the early ages of the concrete when it would be most effective in relieving stresses. The effect of flow cannot now be accepted as anything more than an additional factor of safety. With the Freyssinet method the shortening effects can be measured over any desired period and adjustments made accordingly. In addition, the effects of the superstructure, which to a large extent vitiates the analysis of an arch considered as free, may be determined

*JOURNAL Amer. Concrete Inst., Oct., 1932; *Proceedings*, Vol. 29, p. 57; discussion Feb., 1933, Vol. 29, p. 301.

†"Application of Freyssinet Method of Concrete Arch Construction to the Rogue River Bridge in Oregon" by Albin L. Gemeny and Conde B. McCullough, published by the Oregon State Highway Commission, Salem, Oregon.

by the Freyssinet method and proper provision made to utilize to the greatest possible extent the additional strength provided by this part of the structure.

Of course, it is not contended that telemeters differentiate between plastic and elastic strains. Stresses are used in our diagrams because engineers generally think in terms of stresses rather than strains. It is fully recognized that the strains measured by telemeters include all strains which occur. The statement that "this seems to indicate that the telemeters recorded in this case only elastic strain" is based upon the fact that the modulus of elasticity obtained by manipulating the structure and that obtained from laboratory specimens were approximately equal. In the structure, an increment of load was used which produced no rotation and therefore no change in shape of the arch axis. The time between the application of the load increment and the reading of the telemeters was not far different from that in the case of the test specimens in the testing machine, thereby largely eliminating the time factor which produces flow.

Mr. Whitney states that a large part of the apparent rib shortening is due to change in the shape of the arch axis. Although there is not enough information to determine accurately the change in shape of the axis, the rotations measured at the quarter point and crown do not appear to indicate that any great part of this shortening could have been due to this cause.

Mr. Whitney states that no cracks could occur in the surface of the ribs because of the small amount of steel. The shrinkage of the shell of the rib was resisted not only by the steel but also by the more slowly shrinking interior portions of the rib. The existence of surface cracks in mass concrete may be observed in numerous structures even where there is no steel present. The case of the columns tested at the University of Illinois is not analagous to that of the Rogue River arches because of the difference in the mass of concrete under consideration. It is unfortunate that we did not have telemeters at the arch axis as well as on the extrados and intrados. Such an installation would have thrown light on the above questions. No examination of the rib surface was made for cracks.

The final report on this project has been published as a bulletin by the Oregon Highway Commission.

Temperature Effects on Compressive Strength of Concrete*

BY A. G. TIMMS† AND N. H. WITHEY‡

This investigation was made to supply much needed information on strength of concrete made with present-day normal and high-early strength cements when exposed at different temperatures with particular reference to such conditions as prevail when concreting in winter weather. The most extensive data heretofore available on this subject were those reported in Bull. 81 of the Eng. Exp. Sta., Univ. of Ill., 1915.

MATERIALS

Sand and gravel from Elgin, Ill., were used throughout. The fine and coarse aggregates were oven-dried and screened to the following sizes: 0-No. 28, No. 28-16, No. 16-8, No. 8-4, No. 4-⅜-in. and ⅜-¾-in. These sizes were recombined to give a constant grading, the proportions of each having been fixed by previous trials. The proportions for the fine aggregate were 40 per cent 0-No. 28, 25 per cent No. 28-16, 17 per cent No. 16-8 and 18 per cent No. 8-4 and for the coarse aggregate: 25 per cent No. 4-⅜-in., and 75 per cent ⅜-¾-in.

Laboratory mixture (a mixture of equal parts of 4 brands of normal portland cement) and two high-early strength cements were used. Each cement was used with three water contents: 4½, 6, and 9 gal. per sack of cement.

The mix was varied for each cement and water content to give the same remolding effort[1] and approximately the same slump. The data of the several mixes are shown in Table 1. (Tables 1-4 at end of paper).

PROCEDURE FOR MAKING AND STORING SPECIMENS

Batches of sufficient size to make 14 to 22 specimens, depending upon the number required for the desired curing conditions, were

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1"Studies of Workability of Concrete," by T. C. Powers, JOURNAL Amer. Concrete Inst., Feb. 1932, *Proceedings* v. 28, p. 419.

weighed out and mixed for 2 min. in a Lancirick open-tub mixer of 1¼ cu. ft. capacity. Four specimens were made in separate rounds for each test condition. Slump tests were made for each batch for two of the four rounds. The molds used were 3 by 6-in. cardboard cylinders, paraffin treated. They were supported in frames, in gangs of eight to ten. The molds were filled in three layers, each layer being rodded 25 times with a ¼-in. bullet pointed steel rod. Three to 5 hours after making, wherever possible, the cylinders were capped with neat cement according to A. S. T. M. Standard Methods. Where conditions made it impossible to cap with neat cement, neat gypsum was used. All specimens were capped on the bottom with gypsum except 1-day specimens which were rubbed to a true surface on a rubbing board. Eight to 24 hr. after making, the cylinders were removed from the frames without removing the molds.

Specimens to be exposed to low temperatures were first given a preliminary curing (in the molds but not covered) before being placed in the cold room. The temperature during this preliminary curing was the same as the temperature of the concrete at time of placing; that is, either 70 or 50° F. ± 6° F. Four preliminary curing periods of ¼, ⅔, 1 and 3 days were included. The periods in the cold room were also varied so that specimens were removed from the cold room at 1, 3, 7 and 28 days from the time at which they were made. The warming periods were 0, 2, 4, 6, 21, 25, 27, 62 and 83 days. Specimens for each condition were tested at ages of 3, 7, 28 and 90 days. This gave results for no warming and for various warming periods for each preliminary curing and cold storage condition. For the ¼-day curing at 70° F. specimens were also made to be tested at one day.

The temperatures at which the cold room was maintained were 50, 33 and 16° F. These temperatures were maintained within ± 5° F. except that on two occasions the 33° F. temperature varied up to 40° F. for very short periods. In every case specimens were stripped of their paper molds immediately upon removal from the cold room.

Comparison or "key" specimens were made in the same manner. Some of these were given full moist curing at 70° F. (molds removed at 24 hr.). Others were given partial moist and partial dry curing to correspond to the warming (and drying) periods of the specimens exposed at low temperatures. Table 2 gives the compressive strength of the "key" specimens made and cured at 70° F. Six different conditions of curing are shown with ages at test from 1 day to 3 months. The first consists of the standard moist room curing in which the specimens were 1 day in molds and then removed from molds and placed in moist room until tested. In the second curing condition

the specimens were in the molds 1 day, then removed from molds, placed in moist room for 2 days and then removed to air of laboratory at 40 to 60 per cent relative humidity until time of test. The other 4 curing conditions consisted of initial curing in the molds for 1, 3, 7 and 28 days respectively followed by air storage at 70° F. and 40 to 60 per cent relative humidity until time of test.

Concrete was placed at 70 or 50° F. (±6°) as shown in the tables. For the 50° F. temperature, the molds, cement, aggregates and water were precooled before mixing. The temperature of precooling was determined by preliminary trials so that with the warming effect of the mixing action the desired temperature at time of placing was obtained.

Before testing, all specimens, including those cured in the moist room, were soaked in water at 70° F. to bring them to the same condition as regards moisture content and temperature. The specimens tested at 1 day were soaked 1 hr. and those tested at later ages were soaked 2 to 3 hr. This soaking constituted the only warming for those specimens tested immediately upon removal from the cold room. Experience showed that this period in water was sufficient thoroughly to remove the chill from the specimens.

CEMENT FACTOR AND CONSISTENCY

From Table 1, which gives the mix, slump, and cement factor of the concrete, it will be noted that different cement contents are shown from the threc cements at each water content. This reflects the difference in workability and water requirements of the three cements. The procedure as noted above was to adjust the mix until constant consistency and remolding effort were obtained. In general, it will be noted that with high-early-strength cement No. 1 more cement was required for a given water content and consistency than with cement No. 2, but inspection of the values in Table 2 show that cement No. 1 gave higher strength at a given water content. It is interesting to note that when the concretes from these two high-early strength cements are compared on the basis of equal cement content they give essentially the same strengths.

PRINCIPAL RESULTS OF THE TESTS

Complete strength data for these tests are given in Tables 2 and 3. Percentage values derived from these strengths are shown in Table 4. Curves illustrative of part of the data are given in Fig. 1 to 8.

For the first two curing conditions in Table 2 each value shown for ages of 1 to 7 days is the average of 6 and in some cases 12 specimens. The 28-day and 3-month strengths are in general the average of 4

specimens. Not all the 28-day and 3-month values were available at the time of printing this paper. Two each of the specimens for standard moist curing and for 3 days moist curing were made at the beginning, middle and end of the fabricating period. For the other curing methods in Table 2 and all of the tests in Table 3 the values shown are the average of 4 specimens.

From Table 2 it will be seen that moist curing gives the highest strengths at all ages after 7 days for the high early strength cements, and after 3 days for the normal laboratory cement. In general, the specimens removed from molds and cured in air at about 50 per cent relative humidity showed some falling off in strength at the 3-month period due to drying. This study of the effect of storing in the molds and subsequent storage in air with molds removed was necessary properly to interpret the results of tests of specimens cured at low temperatures and then allowed to dry at 70° F.

Table 3 gives the strength data for the 3 cements, 3 water contents, 2 temperatures of placing, 4 periods of initial curing, 3 temperatures of exposure, and 4 periods of subsequent warming. The 2 temperatures of making were 70 and 50° F. The strength values in bold-face type in the table are for specimens tested 2 to 3 hr. after removal from the cold room. All other values are for specimens given a subsequent warming in air at 70° F. and 50 per cent relative humidity after removal from the cold room.

Many studies and comparisons can be made from the values in Table 3. Of particular interest and significance are the very beneficial effects shown by extending the initial curing periods at 70° F. For example, concretes made with both normal and high-early strength cements and stored at 33° F. after 1 day initial curing at 70° F. had 50 to 100 per cent more strength at 3 days than similar concrete given an initial curing of only ¼ day.

RESULTS FOR NORMAL PORTLAND CEMENT

The data can be best studied from the figures. Fig. 1 to 3 show age-strength curves for concrete made at 70° F. with the laboratory mixture. Fig. 1 is for concrete given a preliminary curing of ¼ day at 70° F. In Fig. 2 the preliminary curing at 70° F. was 1 day and in Fig. 3 it was 3 days. The data for ⅔-day preliminary curing were not plotted because they were approximately the same as those for the 1-day preliminary curing period. The 9 diagrams in each figure represent 3 exposure temperatures (50, 33 and 16° F.) and 3 water contents (4½, 6 and 9 gal. per sack of cement). On each diagram the upper solid curve shows the age-strength relationship for the given

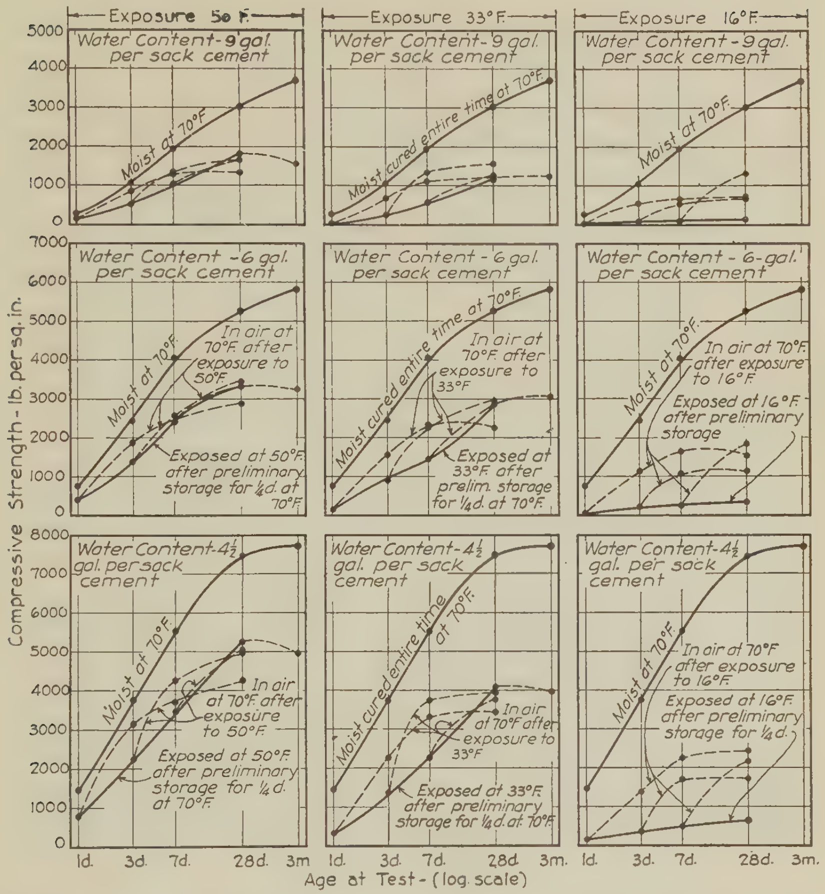

FIG. 1—EFFECT OF EXPOSURE TEMPERATURE ON COMPRESSIVE STRENGTH OF CONCRETE MADE AT 70° F., WITH LABORATORY MIXTURE AND GIVEN ¼D. PRELIMINARY STORAGE—DATA FROM TABLES 2 AND 3

water content based on specimens moist cured at 70° F. The lower solid curve represents the age-strength relationship for concrete tested 2 to 3 hr. after removal from the cold room. The dash-line curves represent the strengths of concrete given a warming treatment in air at 70° F. and 50 per cent relative humidity, upon removal from the cold room.

It will be noted that the strengths shown for concrete stored in the cold room at low temperatures for the entire time diminish as the temperature decreases. The concretes show considerable strength

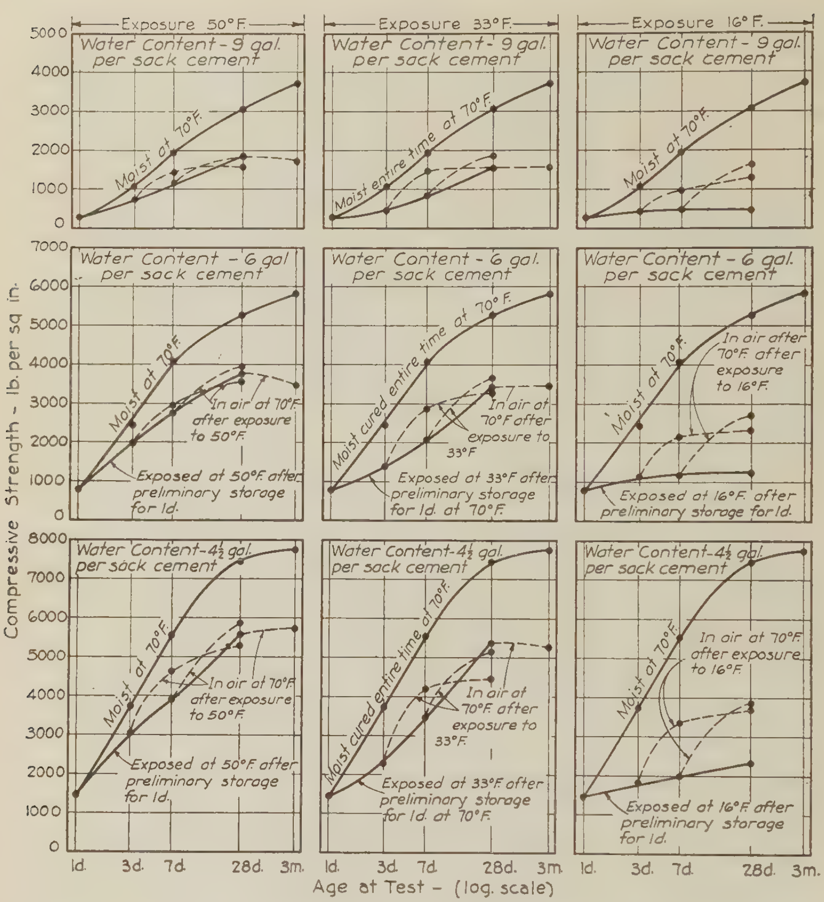

FIG. 2—EFFECT OF EXPOSURE TEMPERATURE ON COMPRESSIVE STRENGTH OF CONCRETE MADE AT 70° F., WITH LABORATORY MIXTURE AND GIVEN 1D. PRELIMINARY STORAGE—DATA FROM TABLES 2 AND 3

when cured at 50 and 33° F. Even at the low temperature of 16° F. surprising strength was shown by the richest mix. For example, in Fig. 1 the concrete containing 4½ gal. of water per sack of cement, stored all but the first 6 hr. at 16° F. had a strength at 28 days of nearly 700 lb. per sq. in. Caution must be exercised in reference to such values, however, as they may lead to false sense of security as to the actual load-carrying ability of the concrete. As pointed out below the increase in strength due to warming in dry air is relatively

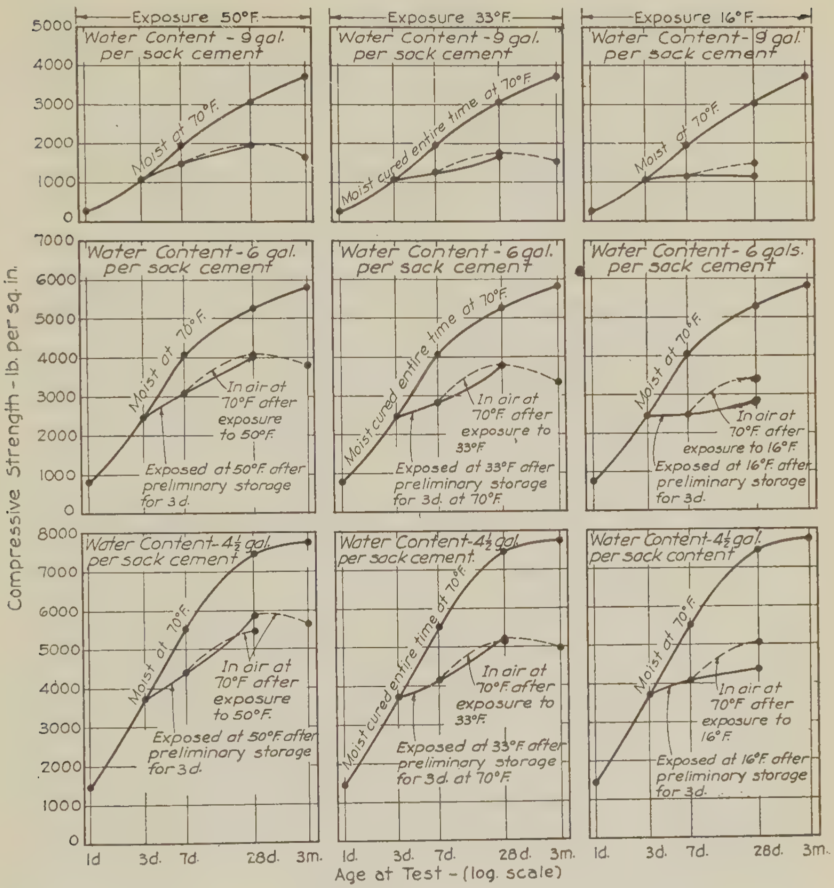

Fig. 3—Effect of exposure temperature on compressive strength of concrete made at 70° F. with laboratory mixture and given 3d. preliminary storage—data from tables 2 and 3

small. Concrete made with 9 gal. of water per sack of cement, and given similar treatment, had practically no strength at 28 days.

The strength of concrete exposed at 50° F. was influenced only slightly by the subsequent warming period in air at 50 per cent relative humidity. The increases in strength due to warming were somewhat greater for concrete exposed at 33° F. and considerably greater for concrete exposed at 16° F. A noteworthy point in Fig. 1, especially for concrete stored at 50° F., is the fact that, due to drying, concrete removed from the molds at 1, 3, or 7 days and stored in air at 70° F.

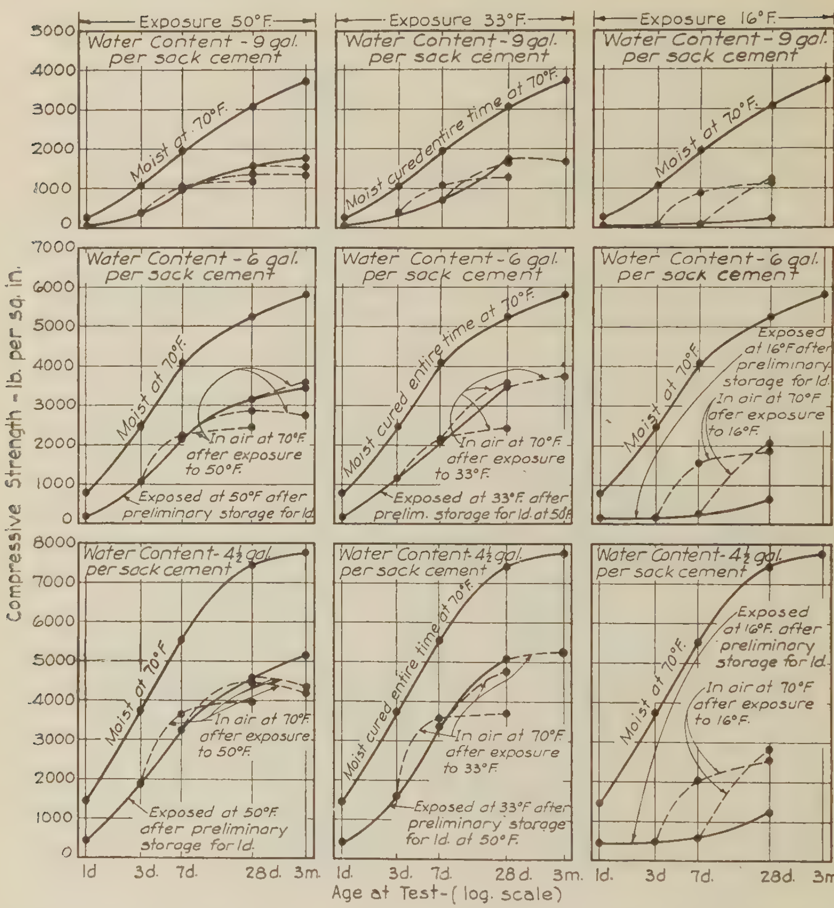

FIG. 4—EFFECT OF EXPOSURE TEMPERATURE ON COMPRESSIVE STRENGTH OF CONCRETE MADE AT 50° F., WITH LABORATORY MIXTURE AND GIVEN 1D. PRELIMINARY STORAGE AT 50° F.—DATA FROM TABLES 2 AND 3

often had lower strengths at 28 days than concrete kept constantly at 50° F. in the molds. This is explained by the fact that the concrete in the cold room was protected by the molds from excessive drying. In many cases even at low temperatures a gain in strength occurred, while the concrete in the air at 50 per cent relative humidity without the protection of molds dried out and at later ages showed little gain in strength and in some cases a slight retrogression.

The curves in the diagrams of Fig. 2 and 3 are similar to Fig. 1 except that they bring out in a striking manner the benefit of additional

preliminary curing at 70° F. A comparison of Fig. 1 with Fig. 2 and 3 shows that in the case of the specimens stored at 50 and 33° F., the benefits derived from additional curing between 1 and 3 days was not as great as between ¼ day and 1 day. This indicates that for these storage temperatures the critical initial curing period for the concrete in the 3 by 6-in. cylinders made from the laboratory mixture was between ¼ day and 1 day. When exposed at 16° F. the concrete was greatly benefited by extending the initial curing period at 70° F. to 3 days.

Fig. 1, 2 and 3 lend themselves readily to the making of comparisons between concretes of different water content and concretes of different preliminary storage. For example, a 9-gal. concrete given 3 days preliminary storage at 70° F. and then stored at 33° F. had a 28-day strength of 1600 lb. From the curves it can be seen that about the same strength was obtained at 7 days with the 6-gal. concrete given only ¼ day initial protection.

Fig. 4 represents specimens made at 50° F. and is similar to Fig. 2 showing specimens made at 70° F. Both are based on tests with the laboratory mixture and a preliminary storage period of 1 day. The principal difference between the curves in the two figures is that the strengths in Fig. 4 are lower. Interesting comparisons which show the effect of temperature of placing can be made between the curves in Fig. 4 and those in Fig. 1, 2 and 3. Such comparisons show very clearly the advantage of placing at a temperature of 70° F. in preventing serious losses in strength for concrete exposed at 16° F.

It is of interest to note here that concrete specimens placed at 50° F. and exposed to temperatures as low as 16° F., even with as little as 1-day preliminary curing at 50° F., showed no imprints of ice crystals after testing.

RESULTS FOR HIGH-EARLY STRENGTH CEMENTS

Fig. 5, 6 and 7, constructed from the data for high-early strength cement No. 1, are similar to Fig. 1, 2 and 3. Similar figures for the other high-early strength cement have been omitted to conserve space.

The curves in the various diagrams for the high-early strength cement are similar to those of the laboratory mixture except that they show higher strengths at given ages. This is explained by the fact that the high-early strength cements have a rapid gain in strength at early ages so that when the concretes were subjected to the low temperature they already had developed high initial strengths.

The same comparisons can be made between the values in Fig. 5, 6 and 7 as were made for those in Fig. 1, 2 and 3. It is surprising that

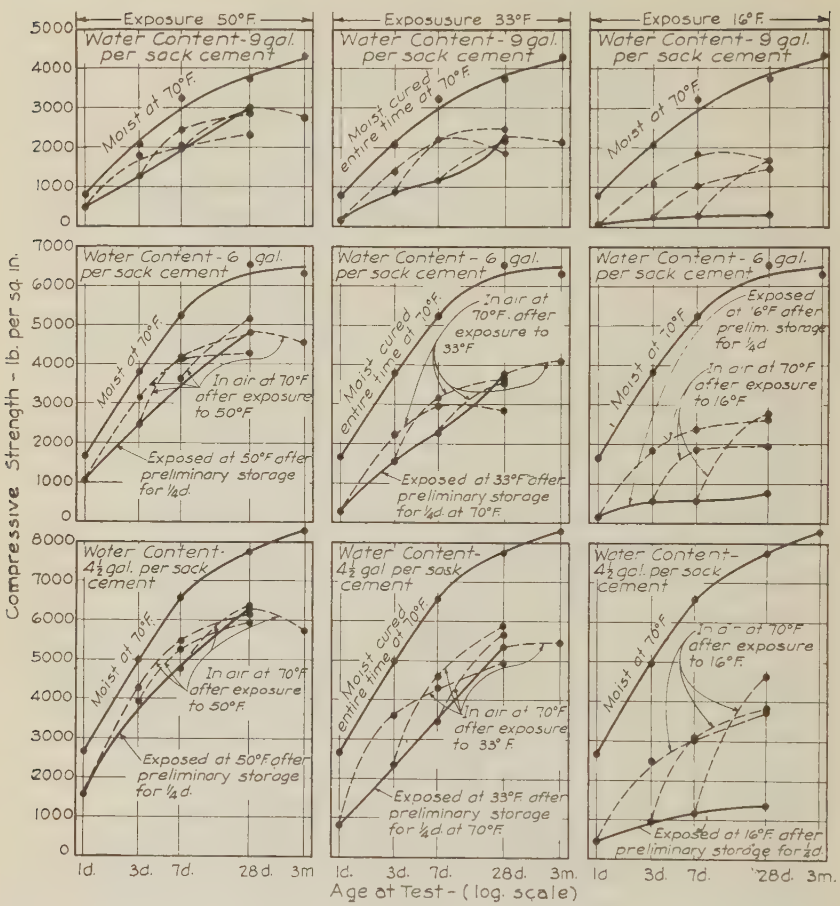

FIG. 5—EFFECT OF EXPOSURE TEMPERATURE ON COMPRESSIVE STRENGTH OF CONCRETE MADE AT 70° F., WITH HIGH EARLY STRENGTH CEMENT NO. 1 AND GIVEN ¼D. PRELIMINARY STORAGE AT 70° F.—DATA FROM TABLES 2 AND 3

both the normal and high early strength cements, when given 1 and 3 days initial curing at 70° F., showed a slight gain in strength during exposure to a temperature of 16° F.

SUMMARY OF STRENGTHS FOR LABORATORY MIXTURE ON A PERCENTAGE BASIS

Table 4 gives data derived from Tables 2 and 3 on the basis of percentages of the strength at 28 days of specimens made with the laboratory mixture and moist cured at 70° F. the entire time. In Fig. 8 the

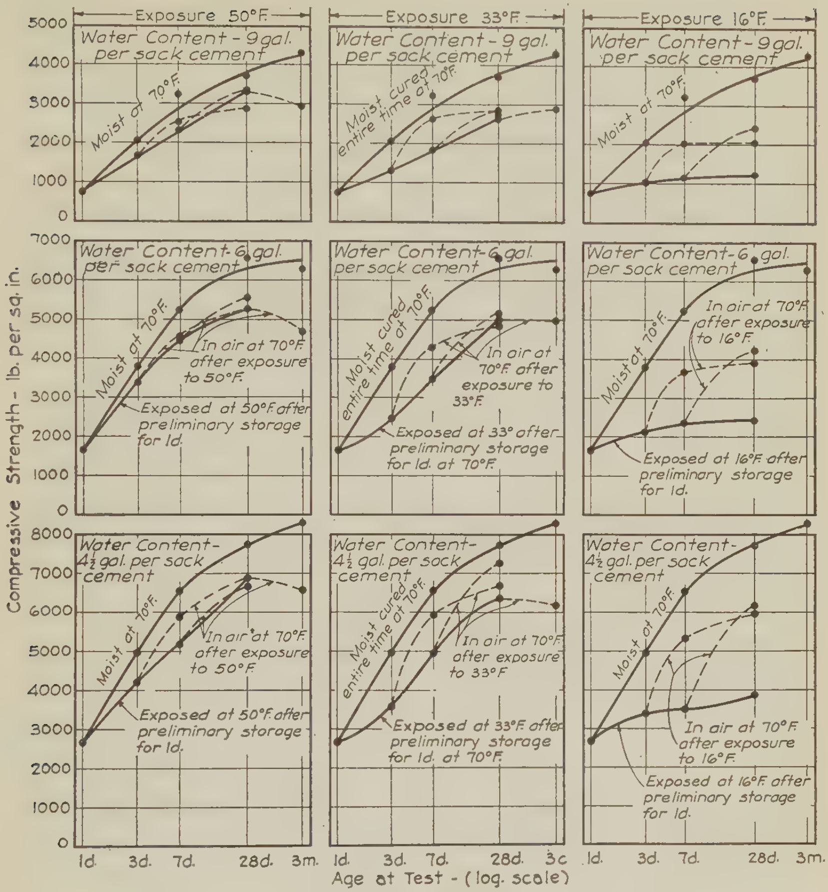

FIG. 6—EFFECT OF EXPOSURE TEMPERATURE ON COMPRESSIVE STRENGTH OF CONCRETE MADE AT 70° F., WITH HIGH EARLY STRENGTH CEMENT NO. 1 AND GIVEN 1D. PRELIMINARY STORAGE AT 70° F.—DATA FROM TABLES 2 AND 3

percentages for concrete of 6-gal. per sack water content and placed at 70° F. have been plotted. This figure consists of 3 diagrams, representing initial curing periods of ¼, 1 and 3 days. The dotted curve in each diagram is for concrete cured 1 day in mold and 2 days moist, then in air (at 70° F. and 50 per cent relative humidity) with molds removed until time of test. Similar figures can be constructed for water contents of 4½ and 9 gal. per sack of cement.

The values used in this diagram are based on concretes tested immediately upon removal from the cold room. It will be noted that

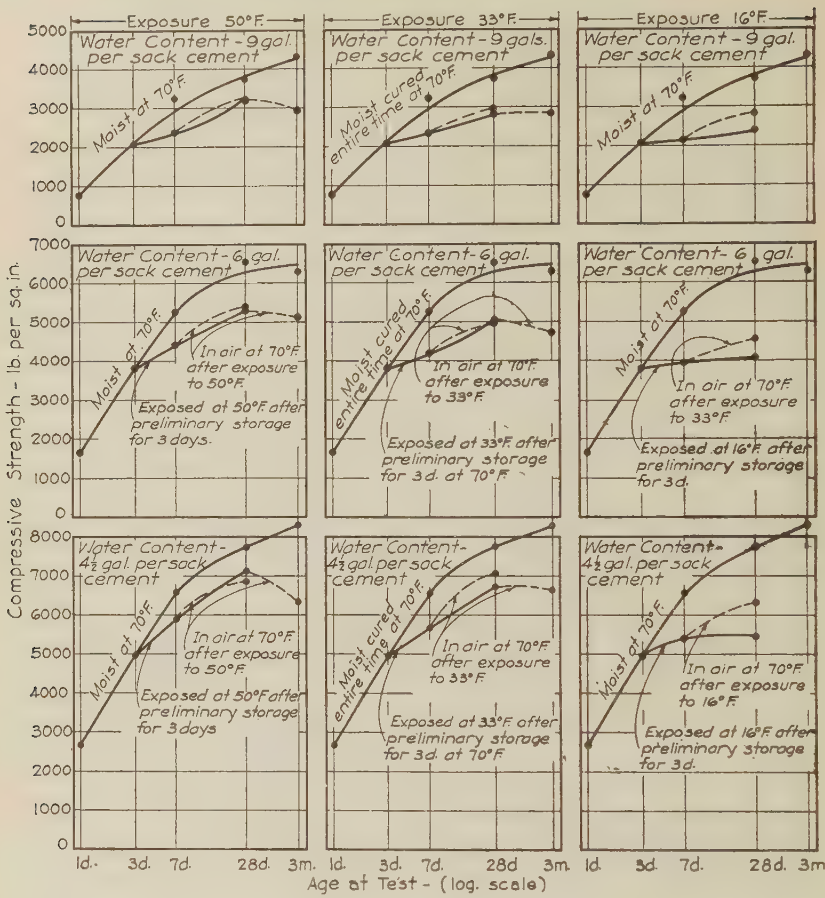

FIG. 7—EFFECT OF EXPOSURE TEMPERATURE ON COMPRESSIVE STRENGTH OF CONCRETE MADE AT 70° F., WITH HIGH EARLY STRENGTH CEMENT NO. 1 AND GIVEN 3D. PRELIMINARY STORAGE AT 70° F.—DATA FROM TABLES 2 AND 3

the concrete exposed to 16° F. increased but little in strength with age, while the concrete exposed at 33 and 50° F. showed appreciable increases. This diagram also brings out the beneficial effect of extending the initial curing period. It will be noted that concrete cured at 50° F. when tested at 28 days had about the same strength as 70° F. moist cured concrete tested at 7 days.

CONCLUSIONS

Following are some of the significant conclusions from the tests:

(1) The rate of hardening of the concrete following a given initial

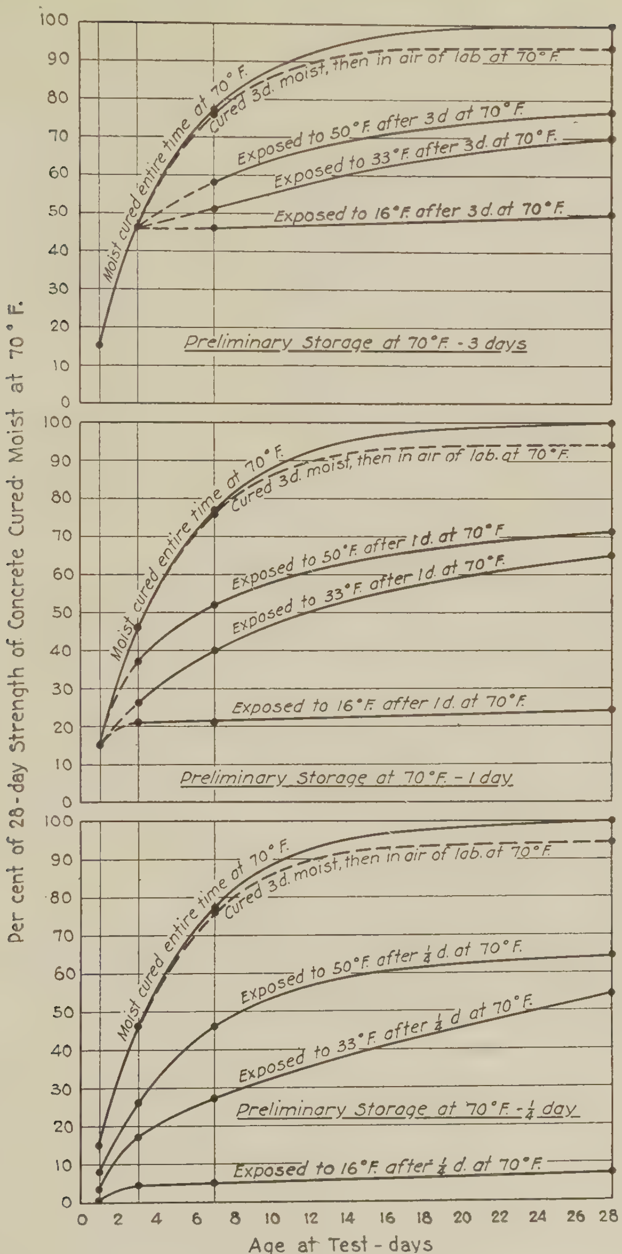

FIG. 8—RELATIVE STRENGTH OF CONCRETE AS INFLUENCED BY STORAGE TEMPERATURE—DATA FROM TABLES 2 AND 4—DATA FOR LABORATORY MIXTURE ONLY. WATER CONTENT: 6 GAL. PER SACK OF CEMENT

treatment was dependent on the temperature of exposure. The 28-day strengths obtained with storage temperatures of 50 and 33° F. were, in general, from 50 to 75 per cent of those obtained with concrete moist cured at 70° F. The rate of gain in strength with age was less for concrete exposed to 33° F. than to 50° F. Even at 16° F. the richest mix concrete showed a definite gain in strength

(2) Subsequent warming of concrete exposed to temperatures of 50 and 33° F. was not of much benefit in improving the later strengths when no provision was made to supply moisture to further the curing action. The greatest benefit from warming occurred with the concrete exposed to 16° F.

(3) The importance of the duration of the period of initial curing is brought out in a striking manner by the tests. The indications are that when the temperature of exposure is from 33° to 50° F., the initial curing period at 70° F. should be at least 3 days for normal cement and at least 1 day for high early strength cement. When the temperature of exposure is below freezing these minimum initial curing periods should be increased depending on the strength required for safety. For concretes exposed to temperatures below freezing, the strength at any time after the period of initial curing depends primarily on the strength developed during the initial curing period.

(4) Comparisons of the strengths of concrete made at different temperatures show the danger of placing concrete having a temperature less than 70° F. where it is to be exposed to temperatures below freezing.

(*Tables 1-4 appear on pages 173-180*)

For such discussion of this paper as may develop, readers are referred to the JOURNAL *for September, 1934* (*Proceedings, Vol. 31*). *Discussions should be available to the Secretary by June 1, 1934.*

TABLE 1—QUANTITIES OF MATERIALS AND WORKABILITY MEASUREMENTS

Weight of sand, dry rodded, 112 lb. per cu. ft.
Weight of gravel, dry rodded, 106 lb. per cu. ft.
Absorption of aggregates—0.95 per cent by weight.

Kind of Cement	Net Water Content, Gal. per Sack Cement	Cement Content Sacks per cu. yd.	Mix: By Weight	Mix: By Volume	Slump—in.: Concrete Placed at 70° F.	Slump—in.: Concrete Placed at 50° F.	Remolding Effort*: Concrete Placed at 70° F.	Remolding Effort*: Concrete Placed at 50° F.
Lab. Mixture	4½	7.72	1:1.69:2.56	1:1.42:2.27	2.9	5.7	38	32
	6	5.68	1:2.78:3.33	1:2.33:2.96	1.9	3.1	40	36
	9	4.04	1:4.24:4.53	1:3.56:4.02	1.6	2.6	39	35
High Early Strength Cement No. 1	4½	8.68	1:1.40:2.13	1:1.18:1.89	2.6	5.8	41	33
	6	6.07	1:2.52:3.03	1:2.11:2.69	2.6	7.3	38	20
	9	3.81	1:4.56:4.88	1:3.83:4.33	1.2	1.5	39	49
High Early Strength Cement No. 2	4½	7.43	1:1.78:2.70	1:1.49:2.40	3.0	6.4	41	30
	6	5.26	1:3.08:3.70	1:2.58:3.28	2.9	5.5	39	38
	9	3.65	1:4.85:5.18	1:4.07:4.60	4.9	6.5	40	52

*"Studies of Workability of Concrete," by T. C. Powers; JOURNAL Amer. Concrete Inst., Feb., 1932; *Proceedings* Vol. 28, p. 419.

TABLE 2—COMPRESSIVE STRENGTHS OF 3 BY 6-IN. CONCRETE CYLINDERS MADE AND CURED AT 70° F.

Each value is the average of at least 4 specimens and in some cases as many as 12 specimens. One-day specimens soaked 1 hr., all others 2 to 3 hr. before testing.

Compressive Strength—lb. per sq. in.

Time in Molds in Air of Lab. Days	Curing Condition after Removal from Molds	Laboratory Mixture (Lot 11788) 1d.	3d.	7d.	28d.	3m.	High Early Strength Cement No. 1 (Lot 11789) 1d.	3d.	7d.	28d.	3m.	High Early Strength Cement No. 2 (Lot 11817) 1d.	3d.	7d.	28d.	3m
Net Water Content—4½ Gal. per Sack Cement																
		Cement Content 7.72 Sacks per cu. yd.					Cement Content 8.68 Sacks per cu. yd.					Cement Content 7.43 Sacks per cu. yd.				
1	Moist Room	1460	3710	5510	7450	7740	2660	4950	6550	7710	8280	2400	4400	5740	6520	6600
1	2d. moist, then in air			5240	6200	5730			6490	7060	6420			5960	6480	6060
1	Air of Lab.		3120	4080	4550	4190		4540	5890	6210	5690		4700	5420	5850	5320
3	"		3950	4940	5160	4790		5520	6640	6600	5950		5040	5610	5570	5250
7	"			5070	5620	5330			6020	6620	5630			5870	6240	5780
28	"				5910	5620				6730	6720				6330	6000
Net Water Content—6 Gal. per Sack Cement																
		Cement Content 5.68 Sacks per cu. yd.					Cement Content 6.07 Sacks per cu. yd.					Cement Content 5.26 Sacks per cu. yd.				
1	Moist	780	2430	4080	5290	5800	1670	3790	5240	6560	6300	1390	2960	3870	4460	4840
1	2d. moist, then in air			4020	4980	4380			5430	5890	5740			4280	4580	4210
1	Air of Lab.		1990	2880	3130	2820		3450	4430	4830	4320		2780	3620	3940	3480
3	"		2580	3610	3730	3280		3900	4640	4910	4370		3270	3570	3880	3470
7	"			3670	4390	4040			5090	5320	4440			3640	4140	3610
28	"				4640	4450				5630	5040				4230	3780
Net Water Content—9 Gal. per Sack Cement																
		Cement Content 4.04 Sacks per cu. yd.					Cement Content 3.81 Sacks per cu. yd.					Cement Content 3.65 Sacks per cu. yd.				
1	Moist	290	1060	1930	3040	3720	780	2050	3210	3710	4310	510	1280	1980	2430	2550
1	2d. moist, then in air			1690	1960	1860			3140	3590	3330			2020	2320	2040
1	Air of Lab.		850	1270	1440	1340		1810	2420	2510	2320		1150	1450	1560	1350
3	"		1100	1540	1500	1580		2180	2760	2750	2580		1430	1790	1680	1520
7	"			1740	1920	1890			2750	3040	2910			1730	1900	1680
28	"				2240	2130				3490	3150				2080	1750

TABLE 3—COMPRESSIVE STRENGTHS OF 3 BY 6-IN. CONCRETE CYLINDERS EXPOSED TO DIFFERENT TEMPERATURES

Strength values in bold-face type in the table are for specimens tested 2 to 3 hr. after removal from the cold room. All other values are for specimens warmed in air at 70° F. and about 50% relative humidity after removal from the cold room.

The duration of the warming period may be calculated by subtracting from the age at test the sum of the values in columns 3 and 5.

Each value is the average of 4 specimens. One-day specimens soaked 1 hr., all others 2 to 3 hr. before testing.

Temp. of Conc. When Placed	Preliminary Storage in Molds before Exposure to Low Temp.		Exposure to Low Temp. in Molds		Compressive Strength—lb. per sq. in.														
					Laboratory Mixture (Lot 11788)					High Early Strength Cement No. 1 (Lot 11789)					High Early Strength Cement No. 2 (Lot 11817)				
	Temp.	Period	Temp.	Period															
°F.	°F.	Days	°F.	Days	1d.	3d.	7d.	28d.	3m.	1d.	3d.	7d.	28d.	3m.	1d.	3d.	7d.	28d.	3.m
Net Water Content—4½ Gal. per Sack of Cement																			
					Cement Content 7.72 Sacks per cu. yd.					Cement Content 8.68 Sacks per cu. yd.					Cement Content 7.43 Sacks per cu. yd.				
70	70	¼	50	¾	**780**	3120	3700	4220	—	1580	4270	5500	6120	—	**1440**	3360	5230	5850	—
				2¾		**2290**	4330	4950	—		**3900**	5260	5940	—		**3090**	5560	6120	—
				6¾			**3490**	5010	—			**4780**	6340	—			**5280**	6530	—
				27¾				**5210**	4940				**6300**	5790				**6030**	5800
70	70	⅔	50	2⅓		**3070**	4510	5280	—		**4300**	5200	6640	—		**3970**	5230	5640	—
				6⅓			**4200**	5530	—			**5670**	6960	—			**4750**	5420	—
				27⅓				**5630**	5130				**6570**	6010				**5870**	5450
70	70	1	50	2		**3080**	4610	5270	—		**4200**	5860	6890	—		**4000**	5280	6340	—
				6			**3870**	5860	—			5130	6650	—			**4980**	6620	—
				27				**5680**	5730				**6850**	6530				**6290**	5650
70	70	3	50	4			**4400**	5490	—			**5840**	6850	—			**4780**	6590	—
				25				**5890**	5620				**7110**	6350				**6270**	5450
50	—	—	50	1	**400**	—	—	—	—	**940**	—	—	—	—	**980**	—	—	—	—
				3		**1840**	3660	3960	—		**3350**	5840	6270	—		**3360**	5020	6210	—
				7			**3280**	4420	4200			**5040**	6890	6120			**5350**	6090	5770
				28				**4550**	4380				**6540**	6560				**5920**	5660
				90					**5110**					**7090**					**6460**

°F.	°F.	Days	°F.	Days	1d.	3d.	7d.	28d.	3m.	1d.	3d.	7d.	28d.	3m.	1d.	3d.	7d.	28d.	3m.
Net Water Content—6 Gal. per Sack of Cement																			
					Cement Content 5.68 Sacks per cu. yd.					Cement Content 6.07 Sacks per cu. yd.					Cement Content 5.26 Sacks per cu. yd.				
70	70	¼	50	¾	**400**	1890	2570	2880	—	**1000**	3190	4150	4270	—	**650**	2420	2800	3440	—
				2¾		**1370**	2550	3340	—		**2440**	4140	4760	—		**2020**	3440	4050	—
				6¾			**2440**	3440	—			**3600**	5130	—			**3140**	3930	—
				27¾				**3360**	3250				**4790**	4560				**3820**	3450
70	70	⅔	50	2⅓		**1820**	3070	3540	—		**3030**	4310	5440	—		**2310**	3250	3950	—
				6⅓			**2820**	4200	—			**4160**	5040	—			**3050**	4010	—
				27⅓				**3890**	3580				**5210**	4830				**3810**	3520
70	70	1	50	2		**1960**	2930	3540	—		**3380**	4560	5280	—		**2490**	3550	4090	—
				6			**2760**	3940	—			**4440**	5580	—			**3030**	4040	—
				27				**3760**	3460				**5240**	4670				**3850**	3860
70	70	3	50	4			**3070**	4080	—			**4400**	5380	—			**3100**	4120	—
				25				**4090**	3730				**5350**	5140				**3970**	3840
50	—	—	50	1	**170**	—	—	—	—	**510**	—	—	—	—	**520**	—	—	—	—
				3		**1020**	2210	2460	—		**2130**	4090	4670	—		**2090**	4090	4230	—
				7			**2130**	2860	2710			**3540**	4740	4330			**3220**	4150	3610
				28				**3120**	3580				**5000**	4710				**4140**	4250
				90					**3430**					**5420**					**4640**
Net Water Content—9 Gal. per Sack of Cement																			
					Cement Content 4.04 Sacks per cu. yd.					Cement Content 3.81 Sacks per cu. yd.					Cement Content 3.65 Sacks per cu. yd.				
70	70	¼	50	¾	**130**	840	1300	1340	—	**410**	1760	2020	2340	—	**230**	1100	1560	1580	—
				2¾		**510**	1390	1680	—		**1260**	2440	2810	—		**900**	1680	1880	—
				6¾			**1040**	1660	—			**1940**	2980	—			**1470**	1910	—
				27¾				**1820**	1550				**2990**	2770				**1990**	1740
70	70	⅔	50	2⅓		**700**	1380	1530	—		**1840**	2390	2910	—		**1110**	1690	1960	—
				6⅓			**1170**	1770	—			**2370**	3000	—			**1570**	2090	—
				27⅓				**1840**	1580				**3120**	2840				**1870**	1880
70	70	1	50	2		**740**	1440	1580	—		**1680**	2520	2890	—		**1040**	1790	2000	—
				6			**1190**	1820	—			**2320**	3310	—			**1560**	2180	—
				27				**1820**	1710				**3320**	2920				**2130**	1930
70	70	3	50	4			**1460**	1990	—			**2380**	3240	—			**1660**	2120	—
				25				**1920**	1630				**3220**	2910				**2110**	1920
50	—	—	50	1	**40**	—	—	—	—	**220**	—	—	—	—	**190**	—	—	—	—
				3		**380**	1010	1150	—		**1120**	2300	2460	—		**990**	1890	1860	—
				7			**980**	1370	1320			**2140**	2750	2580			**1560**	2170	1930
				28				**1550**	1500				**3100**	2700				**2080**	1890
				90					**1730**					**3350**					**2280**

(Table 3—Continued)

°F.	°F.	Days	°F.	Days	1d.	3d.	7d.	28d.	3m.	1d.	3d.	7d.	28d.	3m.	1d.	3d.	7d.	28d.	3m.
Net Water Content—4½ Gal. per Sack of Cement																			
					Cement Content 7.72 Sacks per cu. yd.					Cement Content 8.68 Sacks per cu. yd.					Cement Content 7.43 Sacks per cu. yd.				
70	70	¼	33	¾	**300**	2260	3310	3410	—	**800**	3620	4300	4920	—	**680**	3660	4610	4690	—
				2¾		**1370**	3720	3920	—		**2390**	4590	5830	—		**1980**	4350	5300	—
				6¾			**2230**	3750	—			**3400**	5630	—			**3090**	4970	—
				27¾				**4080**	3950				**5350**	5410				**4690**	4600
70	70	⅔	33	2⅓		**1590**	3860	4450	—		**3030**	5990	6870	—		**2660**	4630	6030	—
				6⅓			**2940**	5080	—			**4640**	6800	—			**4170**	5640	—
				27⅓				**5000**	5500				**6350**	6400				**5820**	5190
70	70	1	33	2		**2300**	4200	4440	—		**3560**	5950	6680	—		**3190**	5290	5720	—
				6			**3460**	5150	—			**4910**	7230	—			**4630**	6140	—
				27				**5320**	5290				**6310**	6160				**5580**	5650
70	70	3	33	4			**4160**	5110	—			**5660**	7040	—			**5100**	6060	—
				25				**5180**	4970				**6720**	6210				**5650**	5970
50	50	⅔	33	2⅓		**1290**	2980	3110	—		**2550**	4710	4890	—		**2370**	4980	4700	—
				6⅓			**2510**	4100	—			**4120**	5440	—			**3830**	5580	—
				27⅓				**3480**	4110				**5180**	5610				**5710**	5000
50	50	1	33	2		**1550**	3550	3670	—		**3180**	5300	5820	—		**2830**	5340	5420	—
				6			**3350**	4750	—			**4960**	6690	—			**4120**	6540	—
				27				**5080**	5200				**6710**	6640				**6060**	6190
50	50	3	33	4			**3020**	4520	—			**4500**	5930	—			**4770**	6550	—
				25				**4800**	4190				**6160**	5880				**5820**	5730

°F.	°F.	Days	°F.	Days	1d.	3d.	7d.	28d.	3m.	1d.	3d.	7d.	28d.	3m.	1d.	3d.	7d.	28d.	3m.
Net Water Content—6 Gal. per Sack of Cement																			
					Cement Content 5.68 Sacks per cu. yd.					Cement Content 6.07 Sacks per cu. yd.					Cement Content 5.26 Sacks per cu. yd.				
70	70	¼	33	¾	**140**	1530	2320	2270	—	**270**	2210	2980	2830	—	**260**	1890	2410	2580	—
				2¾		**900**	2330	2940	—		**1580**	3130	3610	—		**1100**	2760	3430	—
				6¾			**1450**	2890	—			**2290**	3550	—			**2270**	3270	—
				27¾				**2850**	3070				**3770**	4100				**3080**	3330
70	70	⅔	33	2⅓		**870**	2770	3660	—		**1940**	3910	4640	—		**1340**	3290	3950	—
				6⅓			**1920**	3820	—			**3380**	4990	—			**2350**	3930	—
				27⅓				**3490**	3670				**4740**	4500				**3620**	3440
70	70	1	33	2		**1360**	2860	3280	—		**2420**	4300	4880	—		**1890**	3600	4000	—
				6			**2090**	3620	—			**3470**	5130	—			**2640**	4180	—
				27				**3440**	3430				**4990**	4940				**3610**	4040
70	70	3	33	4			**2710**	3780	—			**4150**	4980	—			**3010**	4050	—
				25				**3710**	3310				**5020**	4720				**3740**	3760
50	50	⅔	33	2⅓		**670**	2050	1930	—		**1570**	3290	3090	—		**1410**	3250	3120	—
				6⅓			**1480**	2800	—			**2690**	3740	—			**2370**	3750	—
				27⅓				**3260**	2940				**4250**	4250				**3860**	3440
50	50	1	33	2		**1120**	2140	2420	—		**2300**	4390	4940	—		**1680**	3410	3890	—
				6			**2080**	3540	—			**3930**	4840	—			**2940**	4750	—
				27				**3490**	3770				**5070**	5180				**3990**	4150
50	50	3	33	4			**1780**	3190	—			**3380**	5140	—			**2890**	4420	—
				25				**2890**	3320				**4700**	4840				**3700**	3860

(Table 3—Continued)

°F.	°F.	Days	°F.	Days	1d.	3d.	7d.	28d.	3m.	1d.	3d.	7d.	28d.	3m.	1d.	3d.	7d.	28d.	3m.
Net Water Content—9 Gal. per Sack of Cement																			
					Cement Content 4.04 Sacks per cu. yd.					Cement Content 3.81 Sacks per cu. yd.					Cement Content 3.65 Sacks per cu. yd.				
70	70	¼	33	¾	**50**	660	1100	1150	—	**130**	1380	2200	1850	—	**70**	800	1180	1120	—
				2¾		**250**	1350	1590	—		**840**	2230	2480	—		**380**	1350	1450	—
				6¾			**580**	1280	—			**1150**	2110	—			**790**	1390	—
				27¾				**1220**	1220				**2280**	2160				**1440**	1490
70	70	⅔	33	2⅓		**340**	1430	1930	—		**940**	2390	2770	—		**620**	1590	1820	—
				6⅓			**850**	2080	—			**1800**	2840	—			**1270**	2020	—
				27⅓				**1720**	1800				**2840**	2730				**2050**	1870
70	70	1	33	2		**490**	1470	1520	—		**1300**	2610	2770	—		**790**	1780	1830	—
				6			**840**	1820	—			**1810**	2820	—			**1130**	1890	—
				27				**1540**	1550				**2680**	2840				**1660**	1770
70	70	3	33	4			**1240**	1730	—			**2300**	2910	—			**1440**	1840	—
				25				**1680**	1480				**2810**	2850				**1800**	1710
50	50	⅔	33	2⅓		**210**	810	870			**700**	1810	1800			**570**	1390	1330	
				6⅓			**390**	1420	—			**1390**	1900	—			**1110**	1720	—
				27⅓				**1340**	1430				**1370**	2330				**2020**	1810
50	50	1	33	2		**400**	1080	1290	—		**980**	2280	2250	—		**710**	1580	1810	—
				6			**700**	1680	—			**1800**	3000	—			**1380**	2280	—
				27				**1740**	1690				**3050**	3030				**2310**	2130
50	50	3	33	4			**720**	1480	—			**1870**	2860	—			**1390**	2070	—
				25				**1230**	1500				**2730**	2590				**2050**	2110
Net Water Content—4½ Gal. per Sack of Cement'																			
					Cement Content 7.72 Sacks per cu. yd.					Cement Content 8.68 Sacks per cu. yd.					Cement Content 7.43 Sacks per cu. yd.				
70	70	¼	16	¾	**120**	1360	2240	2430	—	**300**	2490	2950	3780	—	**450**	3000	4550	3990	—
				2¾		**310**	1660	1700	—		**990**	3060	3840	—		**1170**	3190	3870	—
				6¾			**480**	2160	—			**1050**	4650	—			**1270**	3940	—
				27¾				**640**	—				**1360**	—				**1790**	—
70	70	1	16	2		**1830**	3310	3700	—		**3400**	5360	5970	—		**3010**	5150	5560	—
				6			**2000**	3880	—			**3470**	6190	—			**3330**	5780	—
				27				**2320**	—				**3870**	—				**3650**	—
70	70	3	16	4			**4030**	5040	—			**5340**	6310	—			**5070**	6150	—
				25				**4340**	—				**5460**	—				**5280**	—
50	50	1	16	2		**410**	2010	2510	—		**1110**	4190	4770	—		**1060**	3930	4830	—
				6			**520**	2800	—			**1510**	5090	—			**1320**	5110	—
				27				**1210**	—				**2910**	—				**3230**	—
50	50	3	16	4			**1840**	3660	—			**2810**	5730	—			**3030**	5090	—
				25				**2490**	—				**4110**	—				**3750**	—

°F.	°F.	Days	°F.	Days	1d.	3d.	7d.	28d.	3m.	1d.	3d.	7d.	28d.	3m.	1d.	3d.	7d.	28d.	3m.
Net Water Content—6 Gal. per Sack of Cement																			
					Cement Content 5.68 Sacks per cu. yd.					Cement Content 6.07 Sacks per cu. yd.					Cement Content 5.26 Sacks per cu. yd.				
70	70	¼	16	¾	**60**	1110	1660	1530	—	**160**	1810	2390	2620	—	**190**	1920	2970	2400	—
				2¾		**220**	1050	1150	—		**590**	1860	1950	—		**560**	1710	2210	—
				6¾			**240**	1870	—			**590**	2790	—			**530**	2760	—
				27¾				**370**	—				**760**	—				**800**	—
70	70	1	16	2		**1120**	2160	2320	—		**2140**	3690	3900	—		**1670**	3100	4350	—
				6			**1120**	2710	—			**2380**	4250	—			**1790**	3820	—
				27				**1290**	—				**2440**	—				**1740**	—
70	70	3	16	4			**2480**	3350	—			**3950**	4570	—			**3140**	3860	—
				25				**2740**	—				**4050**	—				**3280**	—
50	50	1	16	2		**180**	1580	1890	—		**570**	2400	3150	—		**640**	2720	3160	—
				6			**280**	2010	—			**780**	3060	—			**710**	3370	—
				27				**620**	—				**1620**	—				**1750**	—
50	50	3	16	4			**1010**	2330	—			**2100**	3760	—			**1700**	3230	—
				25				**1340**	—				**2640**	—				**2280**	—
Net Water Content—9 Gal. per Sack of Cement																			
					Cement Content 4.04 Sacks per cu. yd.					Cement Content 3.81 Sacks per cu. yd.					Cement Content 3.65 Sacks per cu. yd.				
70	70	¼	16	¾	**10**	570	690	730	—	**50**	1060	1870	1690	—	**40**	730	900	810	—
				2¾		**70**	510	650	—		**250**	1010	1410	—		**150**	720	850	—
				6¾			**90**	1310	—			**250**	1660	—			**100**	990	—
				27¾				**110**	—				**300**	—				**200**	—
70	70	1	16	2		**420**	980	1300	—		**1100**	2040	2090	—		**760**	1320	1420	—
				6			**430**	1630	—			**1160**	2420	—			**750**	1670	—
				27				**460**	—				**1270**	—				**830**	—
70	70	3	16	4			**1120**	1480	—			**2150**	2890	—			**1490**	1900	—
				25				**1130**	—				**2410**	—				**1630**	—
50	50	1	16	2		**70**	880	1160	—		**230**	1460	1820	—		**240**	1230	1310	—
				6			**90**	1220	—			**310**	1910	—			**300**	1350	—
				27				**210**	—				**620**	—				**590**	—
50	50	3	16	4			**370**	1240	—			**990**	2000	—			**900**	1540	—
				25				**500**	—				**1360**	—				**1090**	—

TABLE 4—SUMMARY OF DATA ON PERCENTAGE BASIS FOR LABORATORY MIXTURE
(Data Derived from Tables 2 and 3)

Net Water Content Gal. per Sack of Cement	Preliminary Storage in Molds Days	Per Cent of 28-Day Strength of Concrete Cured Moist at 70° F.												
		Age at Test												
		1 Day	3 Days		7 Days			28 Days				3 Mo.		
		Warming Period—Days												
		0	0	2	0	4	6	0	21	25	27	0	62	83
Temperature of Placing—70° F. Temperature of Exposure—50° F.														
4½	¼ at 70° F.	10	31	42	47	58	50	70	67	66	57		66	
	⅔ "		41		56	61		75	74	71			69	
	1 "		41		52	62		76	79	71			77	
	3 "				59			79	74				75	
6	¼ at 70° F.	8	26	36	46	48	49	64	65	63	55		62	
	⅔ "		34		53	58		74	80	67			68	
	1 "		37		52	55		71	75	67			65	
	3 "				58			77	77				71	
9	¼ at 70° F.	4	17	28	34	46	43	60	55	55	44		51	
	⅔ "		23		38	45		60	58	50			52	
	1 "		24		39	47		60	60	52			56	
	3 "				48			63	65				54	
Temperature of Placing—70° F. Temperature of Exposure—33° F.														
4½	¼ at 70° F.	4	18	30	30	50	44	55	50	53	46		53	
	⅔ "		21		39	52		67	68	60			74	
	1 "		31		46	56		71	70	60			71	
	3 "				56			70	69				67	
6	¼ at 70° F.	3	17	29	27	44	44	54	55	56	43		58	
	⅔ "		17		36	52		66	72	69			69	
	1 "		26		40	54		65	68	62			65	
	3 "				51			70	72				63	
9	¼ at 70° F.	2	8	22	19	44	36	40	42	52	38		40	
	⅔ "		11		28	47		57	69	63			59	
	1 "		16		28	48		51	60	50			51	
	3 "				41			55	57				49	
Temperature of Placing—70° F. Temperature of Exposure—16° F.														
4½	¼ at 70° F.	2	4	18	6	22	30	9	29	23	33			
	1 "		25		27	44		31	52	50				
	3 "				54			58	68					
6	¼ at 70° F.	1	4	21	5	20	31	7	35	22	29			
	1 "		21		21	41		24	51	14				
	3 "				46			50	62					
9	¼ at 70° F.	0	2	19	3	17	23	4	43	21	24			
	1 "		14		14	32		15	54	43				
	3				37			37	49					
Temperature of Placing—50° F. Temperature of Exposure—50° F.														
4½		5	25		44	49		61	59	53		69	59	56
6		3	19		40	42		59	54	47		65	68	51
9		1	13		32	33		51	45	38		57	49	43
Temperature of Placing—50° F. Temperature of Exposure—33° F.														
4½	⅔ at 50° F.		17		34	40		47	55	42			55	
	1 "		21		45	48		68	64	49			70	
	3 "				41			65	61				56	
6	⅔ at 50° F.		13		28	39		62	53	37			56	
	1 "		21		39	40		66	67	46			71	
	3 "				34			39	60				45	
9	⅔ at 50° F.		7		13	27		44	47	29			47	
	1 "		13		23	36		57	55	42			56	
	3 "				24			40	49				49	
Temperature of Placing—50° F. Temperature of Exposure—16° F.														
4½	1 at 50° F.		6		7	27		16	38	34				
	3 "				25			33	49					
6	1 at 50° F.		3		5	30		12	38	36				
	3 "				19			26	44					
9	1 at 50° F.		2		3	29		7	40	38				
	3 "				12			16	41					

The Effect of Plastic Flow in Rigid Frames of Reinforced Concrete*

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1. OUTLINE AND DESCRIPTION OF TESTS

This report summarizes the results of tests of a number of reinforced concrete rigid frames which have been subjected to sustained loading and observation for two years. The object was to observe the increase in deflections and deformations due to shrinkage and plastic flow and to determine what effect such changes produced upon the distribution of moments and stresses in such frames. The tests were started in December 1931 and observations have been continued to date.

Ten frames, 5 ft. 9½ in. high and 7 ft. 7 in. wide, consisting of five pairs of duplicate specimens, were used. Two were of a closed rectangular type and the remainder were two legged U-frames, hinged at the base. The general dimensions and reinforcement are indicated in Fig. 1. The variables in the U-frames were the amount of compressive reinforcement and the size of the columns.

The frames were loaded at the one-third points of the horizontal members, using calibrated car springs as a means of measuring and maintaining a continued load. The loads were designed to produce maximum working stresses of 20,000 p.s.i. in the steel and 1400 p.s.i. in the concrete in compression. Properties of the steel and concrete are given in Tables 1 and 2. The average strength of 28-day control cylinders was 3340 p.s.i. The value of n used in computations was 8.6, and was based on the observed modulus of elasticity of 3,460,000 p.s.i. for 28-day cylinders. The concrete proportions were 1:3¼:4:⅔, by weight; for cement, sand, gravel, and water.

The frames were poured in an upright position and were cured under wet burlap until tested. They were made in December, 1931, and placed under load about January 1, 1932, when 28 days old.

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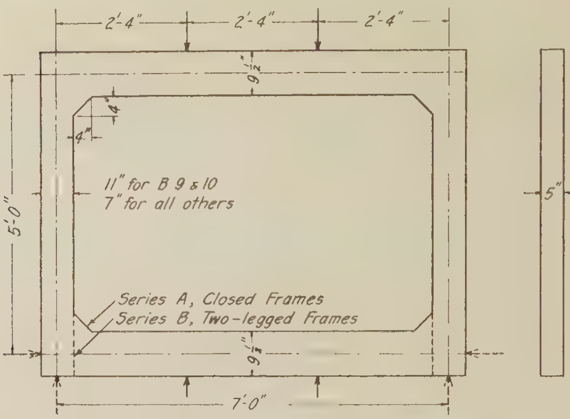


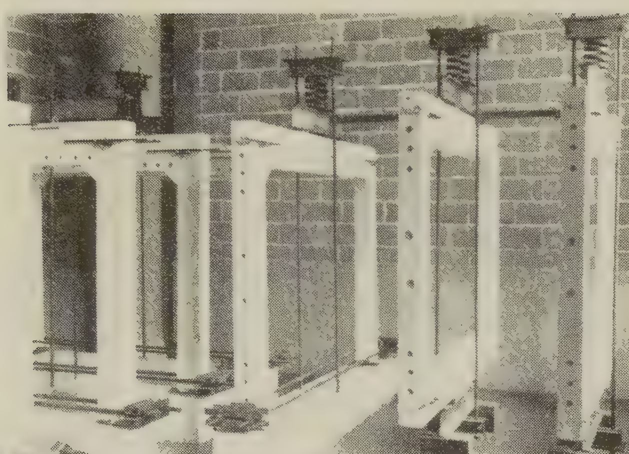
Frame No.	Percent Reinforcement				Stiffness Ratio Beam to Column
	Beam		Column		
	p	p'	p	p'	
A 1 & 2	1.5	0	1.1	0	2
B 3 & 4	1.5	0	1.1	0	2
B 5 & 6	1.5	0.5	1.1	0.4	2
B 7 & 8	1.5	1.0	1.1	0.7	2
B 9 & 10	1.5	0	1.1	0	½

FIG 1—DIMENSIONS AND REINFORCEMENT OF FRAMES

Initial strain readings were taken before and after loading to provide the ordinary values of elastic deformation to be compared with the later observations of deformation under sustained loading. Strains were read at all points of high stress on concrete and on tensile and compressive reinforcement. A series of continuous gage lines was carried across the computed section of inflection for the purpose of noting any movement of the point of inflection due to time effects. Rotation of the corners of the frame was measured by use of the level bar. Deflections were measured in both vertical and horizontal directions by use of Ames dial deflectometers.

The bases of each U-frame rested on small rollers and the horizontal thrust was restrained and measured by means of a car spring. The distance between bases was maintained constant by means of the Ames dial deflectometer. A view of several of the frames under load is seen in Fig. 2, and the method of taking readings with the level bar gage is illustrated in Fig. 3.

Readings were taken at fairly close intervals during the first four months of loading; then, with a decided decrease in the rate of flow, the readings were taken less frequently. Each time readings were taken the springs were adjusted to take up the movement permitted by yielding of the concrete frame.



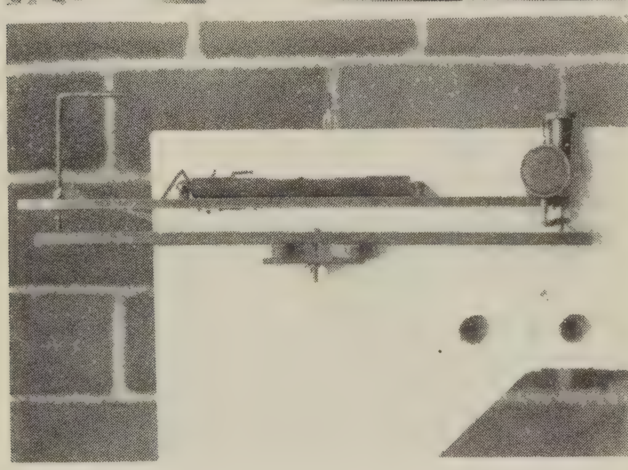


FIG. 2—SEVERAL FRAMES UNDER SUSTAINED LOAD

FIG. 3—LEVEL BAR GAGE FOR MEASURING ANGLE CHANGES

2. GENERAL RESULTS OF TESTS

From 50 to 75 gage lines for strain, deflection or rotation were read on each of the ten frames at each time of observation. So far about 20 complete sets of observations have been taken.

The frames were stored in a small closed room throughout the test and temperature and humidity records were kept. The average relative humidity over the 2-year period was about 60 per cent, with a maximum variation of 18 per cent from this value. The average

temperature was about 73° F.; two or three variations from the average exceeding 6°. Strain measurements have been corrected to correspond to a uniform temperature of 73°. For deflections and rotations no corrections were necessary.

TABLE 1—PROPERTIES OF REINFORCING STEEL

All bars were deformed rounds

Diam. of Bar In.	No. of Tests	Yield Point p.s.i.	Ultimate Strength p.s.i.	Elong. in 8 in. Per Cent	Reduction in Area Per Cent	Remarks
⅝	22	58,200	96,200	17.8	40.6	Used in all frames
½	4	53,100	81,400	21.9	54.0	Comp. reinf., Frames B5-B8
⅜	3	46,700	72,400	24.6	61.0	Comp. reinf., Frames B5-B6

TABLE 2—PROPERTIES OF CONCRETE CONTROL CYLINDERS

Concrete 1:3¼:4, by wt. Universal Cement, torpedo sand, 4-1" gravel. All values in table are average from tests of four 6 by 12 in. cylinders. Curing—28 days in moist closet, remaining time in air. 28-day cylinders tested moist; 90-day cylinders tested air-dry.

Frame No.	Slump	Flow	Comp. Strength, p.s.i.		Modulus of Elasticity, p.s.i.	
			28 Days	90 Days	28 Days	90 Days
A1	6.0	138	3250	4320	3,550,000	4,330,000
A2	7.3	144	3350	4470	3,610,000	4,300,000
B3	7.2	150	3230	4640	3,370,000	3,670 000
B4	6.8	159	3220	4370	3,470,000	3,870,000
B5	7.1	152	3260	4370	3,520,000	4,130,000
B6	4.6	128	3650	4530	3,520,000	3,760,000
B7	7.1	144	3290	4440	3,540,000	4,100,000
B8	7.0	143	3450	4400	3,550,000	3,990,000
B9	7.1	140	3180	4200	3,090,000	3,930,000
B10	7.5	144	3530	3980	3,360,000	3,280,000
Ave.			3340	4370	3,460,000	3,940,000

TABLE 3—INCREASE IN STRAINS AND DEFLECTIONS IN 4-MONTH AND 2-YEAR PERIOD

Frame No.	Period of Loading	Ratio of Total Movement to Initial Elastic Movement						
		Unit Strain					Deflection	Rotation
		Tensile Reinf.	Comp. Reinf.		Concrete			
		Midspan	Midspan	Top of Column	Midspan	Top of Column	Midspan	Corners
A1-2	4 mo.	1.5	...	...	4.4	4.9	3.2	3.2
B3-4	4 mo.	1.6	...	...	5.0	7.6	4.0	3.8
B5-6	4 mo.	1.5	4.4	3.0	5.2	3.9	3.0	3.0
B7-8	4 mo.	1.5	5.3	4.1	3.7	3.9	2.8	2.7
B9-10	4 mo.	1.4	...	...	4.2	4.2	2.5	2.3
Ave.		1.5	4.8	3.6	4.5	4.9	3.1	3.0
A1-2	2 yr.	1.6	...	...	5.4	5.8	3.9	3.8
B3-4	2 yr.	1.8	...	...	6.1	8.5	4.6	4.5
B5-6	2 yr.	1.7	5.1	3.3	6.2	4.2	3.6	3.4
B7-8	2 yr.	1.6	5.7	4.4	4.1	4.1	3.3	2.9
B9-10	2 yr.	2.2	...	...	5.3	5.2	3.0	2.6
Ave.		1.8	5.4	3.8	5.4	5.5	3.7	3.4
Ratio $\frac{\text{Av. 4 mo. value}}{\text{Av. 2 yr. value}}$		0.83	0.89	0.94	0.83	0.89	0.84	0.88

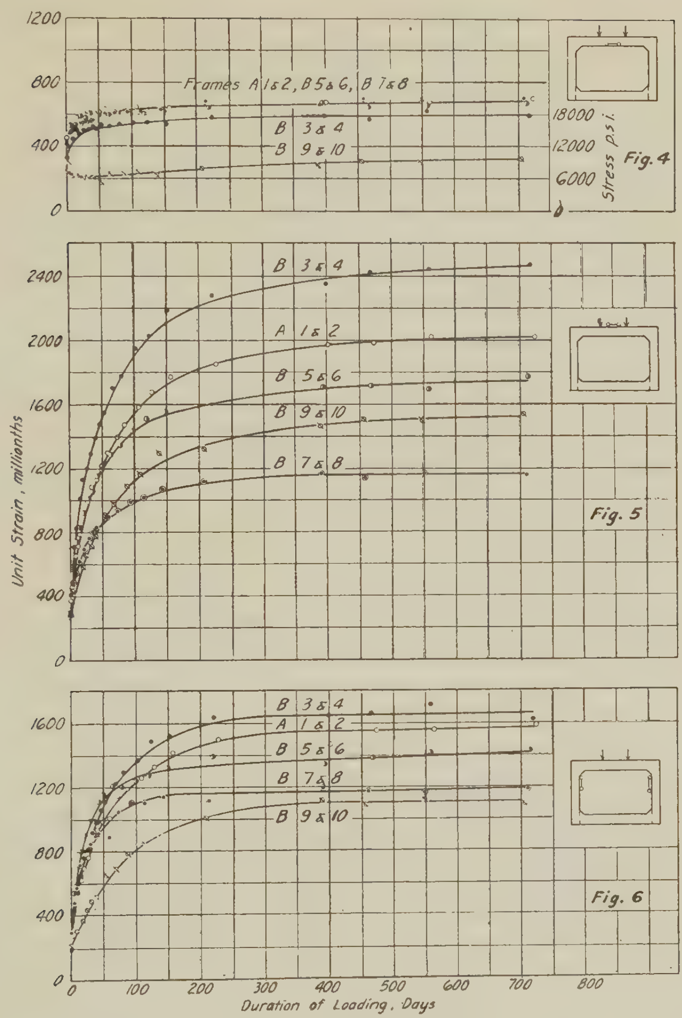


Fig. 4—Tensile strains and stresses in steel at midspan
Fig. 5—Compressive strains in concrete at midspan
Fig. 6—Compressive strains in concrete near tops of columns

Average curves for strains and displacements at critical sections of all frames are presented herewith; these are followed by some typical curves for individual frames. Fig. 4 indicates the variation in average tensile stress in the reinforcing steel at the middle of the top horizontal members of all frames. The increase in stress over the 2-year period

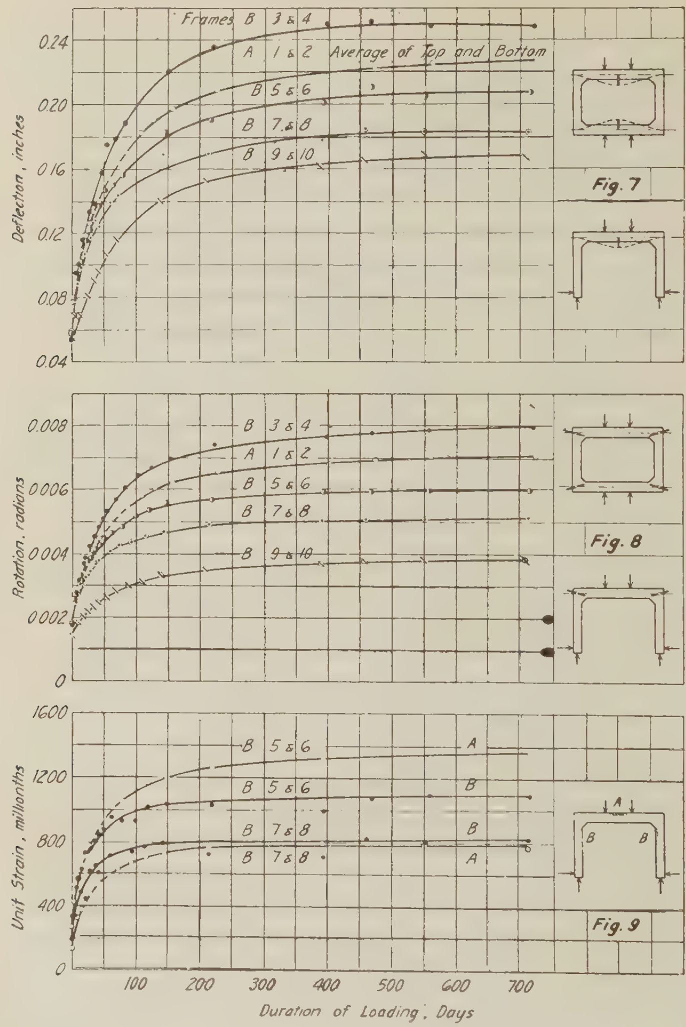


FIG. 7—VERTICAL DEFLECTIONS AT MIDSPAN

FIG. 8—ANGLE CHANGES AT CORNERS OF FRAMES

FIG. 9—COMPRESSIVE STRAINS AND STRESSES IN REINFORCEMENT AT MIDSPAN AND NEAR TOP OF COLUMN OF FRAMES B5-6 AND B7-8

is not very great as compared to other variations observed and may be due to several causes: (a) a gradual extension of cracks in the concrete as noted during the first few weeks of loading, (2) a tensile plastic yielding of the concrete, and (3) a lowering of the neutral axis

and decrease in *jd* with the large flow of the concrete on the compression side of the member. These three sources of change are counteracted to some extent by shrinkage of the concrete. The stresses approach but do not generally exceed the value of 20,000 p.s.i. assumed in the design. It may be noted that in Frames B9 and B10, the moment at midspan did not govern the design load, and hence the strains at midspan are low as shown in Fig. 4. Furthermore, the load first applied on these two frames was in error and was reduced after a few days, as indicated by the drop in the deformation curve.

Fig. 5 presents time-deformation curves for the concrete in compression at midspan of the horizontal members of all frames. The systematic increase of strains to 4 to 6 times the original value is evidently due to plastic flow and shrinkage. It is apparent that a very large part of this increase took place during the first four months of loading. Some of the deformations obtained, 0.0015 to 0.0025, are as great as the ultimate deformation of concrete when loaded rapidly to failure.

The curves of Fig. 6, showing deformations in concrete near the top of columns, are similar to those of Fig. 5, but since the strains shown were not measured at a point of maximum moment, their values are relatively smaller. The final total strain after the 2-year loading is 4 to 8 times the initial value. Fig. 7 shows average curves for vertical deflection of horizontal members at midspan. Here again the readings increased to 3.0 to 4.6 times the initial values during the 2-year loading period. It will be remembered that deflection may be produced by shrinkage in a beam having only tensile reinforcement, though it would not be if symmetrical tensile and compressive reinforcement were present. The compressive reinforcement of Frames B5 to B8 has evidently checked the deflection, while the low value for Frames B9 and B10 is due to a relatively low loading. Curves for horizontal deflections of the vertical legs of all frames (not shown) are similar to those of Fig. 7.

Average curves showing the change in slope, or angular rotation, at the corners of all frames are shown in Fig. 8. These rotations at the end of two years of loading were 2.7 to 4.5 times the values measured at initial loading; this increase was practically identical with the increase in deflection in Fig. 7.

The various curves show that most of the time yield occurred during the first four months of loading and that yielding has practically ceased at the end of two years. Table 3 gives the ratio of final to

initial value of the principal measurements for the four-month and two-year periods.

3. STRESSES IN COMPRESSIVE REINFORCEMENT

Probably the most important effect of time yield is to produce very high stresses in compressive reinforcement. As an example, the stresses in the compressive reinforcement of Frames B5 and B6 are compared with those in Frames B7 and B8 in Fig. 9. The frames were alike except for the compressive reinforcement and were subjected to the same loading. A further comparison of the data from Frames B3 to B8 is provided in the following tabulation, which gives average stresses and strains at the end of two years loading. In this comparison it should be noted that strains on Frames B3-4 are at the surface, while those on compression reinforcement of Frames B5 to B8 are 1½ in. below the surface of the concrete.

Frame No.	At point A, Fig. 9			At Point B, Fig. 9		
	Per Cent Comp. Reinf.	Strain, Millionths	Steel Stress p.s.i.	Per Cent Comp. Reinf.	Strain, Millionths	Steel Stress p.s.i.
B3, B4	0	2460	......	0	1650	......
B5, B6	0.5	1360	40,800	0.4	1080	32,400
B7, B8	1.0	765	22,950	0.7	820	24,600

It is evident that compressive reinforcement greatly reduces the deformation due to time yield and thus permanently stiffens the frame, but it is also true that if too small a percentage is used, the steel is likely to be over-stressed. These results agree very well with the results of other tests in which compressive reinforcement was used, notably the A. C. I. column investigation.[1]

The stiffening effect of the compressive reinforcement is to be noted also in the deflection curves of Fig. 7 and the rotation curves of Fig. 8, wherein the values for Frames B3-4, B5-6, and B7-8 may be compared. It may be concluded that compressive reinforcement, especially in amounts of 1 per cent or more, is effective in preventing large distortions of concrete structures due to time yield.

4. PLASTIC FLOW VS. SHRINKAGE

The test data hitherto presented have embodied the combined effects of shrinkage and plastic flow, as well as a change in the modulus of elasticity of the concrete. No test specimens were made for the purpose of separating the effects of shrinkage and flow, though values of E were determined for the concrete when 28 and 90 days old, as given in Table 2. Two methods of estimating the relative importance

[1]JOURNAL, Amer. Concrete Inst.: Mar. 1931, *Proceedings*, Vol. 27, p. 761-835; Jan. 1932, *Proceedings*, Vol. 28, p. 279-346.

of shrinkage and flow were followed. One consists of a study of strains measured where the stresses were very low, as near the bottom of the columns. Fig. 10 shows strain measurements taken on Frames B3 and B4, at gage lines 4½, 30, and 47 in. above the horizontal thrust line. The two upper gage lines may be used in projecting the values for the bottom line to the thrust line, where flexural stresses should be zero and the direct stress small. Thus the projected value is found from Fig. 10 to be .00040 for the 4-month period and .00054 for 2 years. Corresponding values for Frames B9 and B10 are .00049 and .00060. These values are believed to be closely representative of the shrinkage of the concrete, even though these members contained 1.1 per cent of tensile reinforcement. This steel naturally inhibited the full shortening of the concrete (due to shrinkage) in the outer faces but should not materially affect the free shrinkage of the inner faces.

As a second indication of the probable shrinkage, plain concrete members of similar section and made with the same concrete mixture but with another brand of cement and studied in another investigation showed a shrinkage of 0.00027 after four months and 0.00036 after one year.

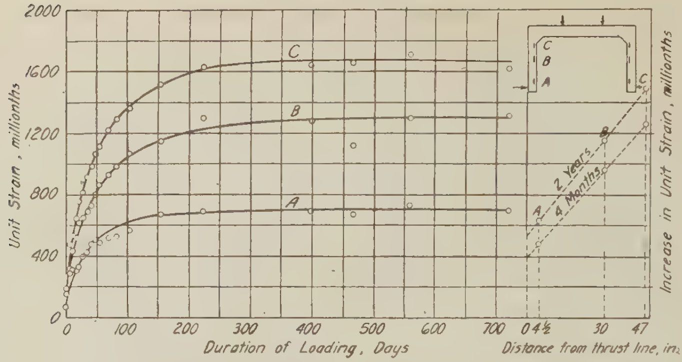
It appears from the foregoing studies that the shrinkage coefficient for these frames was about 0.00045 for the four-month period and 0.00060 for the two-year period. This much shrinkage could occur only at sections not restrained by reinforcement, as, for example, at midspan of Frames A1-2, B3-4, and B9-10 in Fig. 5. Thus of the increase in strain for Frames B3-4, amounting to about 0.0016 at four months, 28 per cent may be attributed to shrinkage, and the remainder to flow. On the same basis the ratio of the shrinkage to the total change is about 35 per cent for Frames A1-2 and 50 per cent for Frames B9-10. These values are in fair accord with the results of the A. C. I. column investigation, wherein the shrinkage for this grade of concrete generally accounted for one-fifth to one-half of the total increase in strain under a one-year sustained loading, with an average proportion of about one-third of the total.

5. DISTRIBUTION OF MOMENTS

The object of this study was to determine whether or not in these statically indeterminate frames a readjustment between positive and negative moments would be produced by time yield in the concrete. If it is assumed, following the tests of Glanville[2] and Davis[3], that the

[2]Glanville, W. H. "Creep or Flow of Concrete Under Load," Tech. Paper No. 12, Bldg. Research Station, Garston, England. See also "Creep of Concrete Under Load," *News Letter*, p. 4, this JOURNAL, June 1933.

[3]Davis, R. E. and Davis, H. E. JOURNAL, Amer. Concrete Inst., Mar. 1931, *Proceedings*, Vol. 27, p. 837-901.



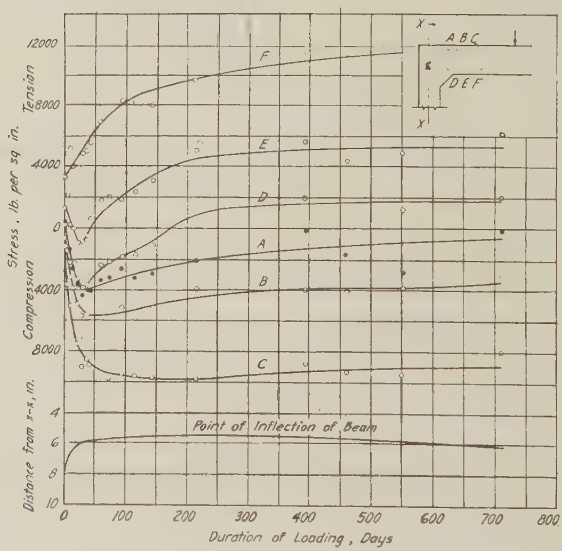


FIG. 10—COMPRESSIVE STRAINS IN COLUMNS OF FRAMES B3-4 AT THREE LEVELS

FIG. 11—STRAINS IN STEEL NEAR POINT OF INFLECTION, FRAMES B7-8

so-called plastic flow or creep at any point in a flexural member is proportional to the intensity of stress at that point, it follows that this creep will be very nearly proportional to elastic deformation (at working stresses) and the effect should be the same as that produced by a decrease in the original modulus of elasticity. Davis has called this decreased modulus the "sustained modulus of elasticity" and has shown that it may reach a value one-half to one-fourth of the original

value for ordinary concrete stored 9 months in air. This concept of time effects is convenient and simple even though it may not be strictly accurate. Bingham and Reiner[4] made tests of hardened neat cement and 1:3 mortar beams, and concluded that the mortar acted as a plastic solid, having a yield value of 65 p.s.i. below which there was no flow under a sustained load, whereas the neat cement acted like a viscous liquid, exhibiting flow at the smallest loads applied and having no perceptible yield value. If concrete is truly plastic, and not viscous, it is evident that the flow will not be proportional to the stress, but if the yield value is no greater than 65 p.s.i., the departure from direct proportionality will not be important when fiber stresses such as 800 p.s.i., or more, are used.

The determination of changes in moment distribution is difficult. The best indication is the observation of the points of inflection in the horizontal members, as shown by a change from tensile to compressive strain, though this is complicated by the presence of shrinkage strains. While the effect of shrinkage is to cause an apparent movement of the point of inflection away from midspan at the top face of the member, it should cause an opposite movement at the bottom face, and the average position determined from the two sets of measurements should be free from shrinkage effects. Fig. 11 shows some typical stress measurements taken near the points of inflection on both top and bottom steel layers of the horizontal members of Frames B7 and B8. Each point shown represents the average of readings at both sides and both ends of the two frames. The deformations measured were small, hence the relative errors are large and the curves rather irregular. For many readings the point of zero stress fell slightly outside the gage points used. The position of this zero point was determined at various ages for the upper and lower steel layers, and the two values averaged. The lower curve of Fig. 11 shows the position of the point of inflection thus determined. The curve indicates that the point of inflection moved outward about 2 in., decreasing the distance to the center line of column from 8 in. to 6 in. during the two-year period. Since the distance from load point to column center line is 28 in., this indicates an increase of positive moment from $\frac{10}{14}\,\frac{Pl}{6}$ to $\frac{11}{14}\,\frac{Pl}{6}$ and a decrease in negative moment from $\frac{4}{14}\,\frac{Pl}{6}$ to $\frac{3}{14}\,\frac{Pl}{6}$. This is a relatively small change and probably not very significant, for

[4]Bingham, E. C. and Reiner, M., "Rheological Properties of Cement and Cement-Mortar-Stone," *Physics*, Vol. 4, Mar. 1933.

while all of the frames show a similar movement of the point of inflection with time, the movements are not consistently in the same direction. In Frames A1 and A2, for example, the point of inflection moved toward midspan during the first four months, then reversed its direction and at the end of two years was slightly outside its initial position.

In the two-legged frames, the measured horizontal thrusts remained very nearly constant for Frames B3-4 and B9-10, and decreased slightly for Frames B5 to B8. The negative moments in the latter, which are proportional to the thrusts, therefore showed a small decrease, of about the same magnitude as noted for B7-8 in Fig. 11. The device used for measuring these thrusts, however, was not as sensitive as it should have been.

While most of the evidence indicates either no moment change or a small decrease in negative moment, it is believed that in view of the smallness and lack of consistency of the moment changes, they may be considered negligible.

6. THE EFFECT OF TIME YIELD UPON DESIGN OF STRUCTURAL MEMBERS

Satisfactory correspondence of design stresses with measured values from rapid loading tests has been amply demonstrated during the last 40 years, but the very marked departures of strains, deflections and angle changes under sustained loading from the initial (or design) values have not been so well recognized and may appear alarming or confusing to many engineers. It is very desirable now to evaluate time yield effects and to determine whether the safety of a reinforced concrete member is changed by such time effects. Probably tests to failure after sustained loading for various periods will be needed to give definite information as to structural safety; however, some tentative conclusions seem justified by the test data already available. As a concrete example, we may examine the effect of a large increase in the modular ratio, n, to perhaps four times the usual design value.

Reinforced Concrete Beam

The principal effect of increasing n is to lower the neutral axis, thus decreasing concrete stresses, and (by reducing jd) increasing the steel stress. For a beam designed by the A. C. I. Standard Building Code and having balanced reinforcement, an increase in n to four times its original value will produce (according to straight-line theory) a decrease in j from 0.875 to 0.80, with a corresponding increase in steel stress of 8.5 per cent. An increase in steel stress from 20,000 to 21,700 p.s.i. would not be serious and it may be concluded that the effect of flow on the strength of such a beam is negligible. The action of flow

in increasing k may justify the use of higher working stresses in concrete.

Double-Reinforced Beam

Current design formulas give results evidently far from actual conditions, though not necessarily unsafe. The A. C. I. Code permits a working stress, f_s', in the compressive steel equal to nf_c, which cannot exceed 12,000 p.s.i. The data of Frames B5 to B8 show that actually these stresses may approach yield point values if the percentage of steel is small. In these frames, the compression steel was embedded approximately 0.4 kd below the surface, so that the initial stresses were well below 12,000 p.s.i., and the effectiveness of the steel was less than it would be in a deeper beam. Recent column tests indicate that even if the yield point were reached, the steel would continue to function if sufficiently embedded in concrete and tied in by stirrups to prevent its buckling. The increase in steel stress relieves the concrete in compression (though the latter initially has a safety factor of 3.5 or more) and it tends to keep jd constant, so that there can be little increase in the tensile steel stress.

Time yield evidently will not greatly affect the safety of double-reinforced beams if the compressive steel is well tied in. The following modification of present design methods apparently would produce a more economical and better balanced design: (a) A minimum limit of about 1 per cent for compressive reinforcement, (b) Use of high elastic limit steel for compressive reinforcement, and (c) Replacement of the present working stress of nf_c in compression steel by an arbitrarily fixed stress of 18,000 or 20,000 p.s.i. The last provision seems reasonable since the ultimate compressive resistance of the beam will combine the yield point stress in the steel and a fiber stress in the concrete (on straight-line theory) of 1.5 f_c', or more.

Axially Loaded Column

The effect of time yield on column action has been covered fully in the Report of Committee 105, JOURNAL, Amer. Concrete Inst., February, 1932. (*Proceedings*, Vol. 28.)

Eccentrically Loaded Column

Current methods of analysis of members under combined stress, requiring solution of a cubic equation when the moment is large, are already cumbersome and certainly are inaccurate when sustained loading is involved. The effect of flow is similar to that in beams in that it results in a considerable reduction in concrete stress and an increase in the steel stress. The use of the current formulas thus pro-

duces a concrete stress higher than the actual stress; furthermore, the design is usually made by keeping this concrete stress within a specified permissible value. The permissible stress in the A. C. I. code has been chosen with some consideration of plastic flow and shrinkage effects, and in some cases is rather high, reaching as great a value as 0.64 f_c'. It appears that both the permissible stress and the method of analysis of eccentrically loaded members need revision and simplification, with particular attention given to time yield effects.

7. LIVE AND DEAD LOADS

Obviously plastic flow will not be produced by loadings of momentary duration, such as wind loads or moving loads on bridges and floors. It will be produced by dead loads and such live loads as are maintained constant during the early life of the structure. Since the tendency to flow decreases rapidly as concrete ages, it is not likely that much flow is caused by live loads. This suggests the desirability of separating live and dead load stresses in designing structures where time yield effects are important. This separation is frequently made, for other reasons. The discussion of Section 6 applies particularly to dead load, or sustained, stresses.

8. SUMMARY

The test frames were made of concrete having a strength of about 3500 p.s.i., and were loaded, after 28 day moist curing, to produce maximum computed working stresses of about 20,000 p.s.i. in tensile steel and 1400 p.s.i. in concrete. Sustained loading was maintained for two years by means of compression springs. The following results are considered important.

1. During the test period large increases were noted in deflections, rotations of joints and compressive strains in concrete and reinforcing steel. The ratio of final to initial values of these quantities ranged roughly from 3 to 6. Values of stress in compressive reinforcement reached 40,000 p.s.i. and compressive unit strains in concrete of 0.0025 were observed.

2. The increase in tensile steel stress was relatively low, with an average ratio of final to initial stress of 1.8.

3. Of the changes noted in the foregoing paragraphs, about seven-eighths of the two-year value occurred in the first four months of loading.

4. While some changes in distribution between positive and negative moments were determined, they were in general so small and irregular that they may be considered negligible

5. Of the total effect of shrinkage and plastic flow, it appears that the latter was more important, comprising two-thirds to four-fifths of the total.

6. Time yield effects apparently will not endanger the safety of reinforced concrete members designed by current methods, if compression steel is properly embedded and tied in. It is believed that present methods of design for eccentrically loaded columns involve the most uncertainty as to time yield effects and need review.

7. While time yield deformations produce distortions of a frame several times that due to initial deformations, such distortions are not generally noticeable or injurious. They can be reduced considerably by the use of compression reinforcement in flexural members.

For such discussion of this paper as may develop, readers are referred to the JOURNAL *for September, 1934 (Proceedings, Vol. 31). Discussions should be available to the Secretary by June 1, 1934.*

Rigid Frame Concrete Bridges*

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INTRODUCTION

The origin of the bridge as a structure is lost in the mists of antiquity. It is reasonable to suppose that the first bridge traversed by our primitive ancestors was a tree which fell conveniently across some stream or chasm. Man, on observing the utility of this structure, was next moved, through exercise of his intelligence, to place a fallen tree at some point on a stream or chasm which he wished to cross. With this act there came the birth of the art of bridge design and construction which has progressed through the ages to its present state of perfection and scientific accuracy.

Bridges, more than any other type of structure or monument, have long held a strong claim on the interest and imagination of mankind. Countless epic poems, narratives, fictional and real romances have been written against the background of bridges—eloquently attesting mankind's appreciation of the value, service and beauty of the bridge and its great contribution to the development of civilization.

This appeal of bridges is not limited to laymen but extends also to the engineers and builders of such structures. Here the scope enlarges from the imaginative and romantic only, to include all of the practical aspects of a problem to be solved and a structure to be built. In almost any good bridge, regardless of type, the purest fundamentals of structural design appropriate to the conditions can be used to the greatest advantage. Hence, it is not strange that among structural engineers the art of bridge design is regarded as somewhat higher and more interesting than ordinary structural design.

Until recently the design of concrete bridges has been somewhat shackled by the persisting influence of stone and steel construction. The stone or masonry influence was felt in the production of arch

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†President, J. W. Beretta Engineers, Inc., San Antonio, Tex.

designs, while the steel influence manifested itself in elaborate precautions to keep concrete beam and girder bridges within statically determinate limitations. This, however, is a natural consequence of structural development and really indicates a transitory period during which designers and builders are getting acquainted with a material. The real indication that a material is about to enter a period of truly scientific usefulness comes when original design methods and details are evolved which are peculiarly suited to it.

STATICALLY INDETERMINATE DESIGN

Therefore, we bridge the recorded development of reinforced concrete as a bridge construction material to consider a specially favorable type of design. Departing from the tradition that different parts of a bridge structure must maintain a separateness of action, we have a new concept which allows us to design a structure in which all parts work together. Thus, we have a form of statically indeterminate design known as continuity. This may take the form of horizontal continuity over multiple spans, restraint in the form of rigid frame continuity, or a combination of both.

Although restraint and continuity are as old as time and statics, the engineering profession has been slow to use these principles due to excessive ingrained conservatism, mathematical fear and possibly laziness. Among the contributions of pioneering engineers to this new design knowledge the names of three men stand out: C. A. P. Turner who several years ago pointed the way to rational continuous concrete slab design; Arthur G. Hayden, designing engineer of the Westchester County Park Commission, New York, who more than any other man deserves credit for the courageous evolution of practical design methods and educational promotion of the idea; Prof. Hardy Cross of the University of Illinois, whose researches and writings on the subject of continuous frames are an invaluable guide in the utilization of the new design methods.

During the last three decades, construction of short span bridges has shown a great predominance of statically determinate types, and it is only in very recent years that the continuous indeterminate types have appeared. Physical evidence of the increasing number of engineer converts to the new methods is shown by the ever growing number of continuous bridges in use. Had their use progressed sufficiently at the turn of the century, millions of dollars could have been saved on the short span bridge structures built as part of our highway and railroad programs.

CLASSIFICATION OF BRIDGES

To obtain the best results with any design method or construction material, it must be used with due regard for its limitations as well as its advantages. It is not the intention of the writer to present structural continuity as a design method of limitless possibilities or reinforced concrete as a panacea for the weaknesses of other structural materials. For the purposes of this paper, therefore, it is pertinent to divide the general subject of bridges into two classifications of long span or monumental structures, and short span or utilitarian types, the latter having maximum spans of about 250 ft.

Long span bridges, with the exception of a few splendidly executed reinforced concrete arch designs, are almost entirely in the province of structural and alloy steels of high strength. In the field of short span bridges, steel as a structural material has played a prominent part through utilization of such popular types as through trusses, pony trusses, girders and rolled beams. Formerly, due to limitations imposed on concrete by statically determinate design and the comparatively crude technique of concrete construction practice, its use in short span bridges has been limited to average spans of 60 ft. or less. Aggregate investment in short span bridges far exceeds that in long span monumental structures. Now, along with improvements in the quality of cement and aggregates, knowledge of concrete mix design and improvements in construction technique, we have new design methods.

Specific uses of short span bridges are widely varied, with some naturally more extensive than others. Their chief uses are to carry highway or railroad traffic over waterways as drainage structures, or as a means of separating these two types of traffic. Each of these uses has specific requirements but they are surprisingly similar. In both classes of structure a minimum number of piers, without sacrifice of span slenderness, headroom and economy, are of paramount importance. Likewise, slenderness of piers is important where problems of water or traffic flow must be met.

DESIGN AND ECONOMIC PROBLEMS OF SHORT SPAN BRIDGES

Relative size and importance of individual short span bridge projects, assure them less care in design, yet the construction problems of smaller bridges are actually as complex and important as of larger ones. Standardization is one of the most expensive construction practices of our day. Long span bridge construction has progressed greatly because of the individual study given each project, while short span bridge construction has not had the benefits of equal research and

development. Small savings in design cost afforded by standardization have too often resulted in large wastes of construction funds.

Economy can be had only by the intelligent use of all advantages of design methods, construction technique and materials which can be applied to any particular project. Materials are selected for availability and low cost, and are distributed through simplified details so as to permit maximum efficiency with every portion of the structure doing its full part. To this can be added such considerations as adaptation to location, requirements of loading, type of crossing, foundation conditions, and construction practices familiar to available labor. Although true engineering knowledge and accurate mathematical analysis will lend symmetry and pleasing proportion to a bridge, this is not sufficient; and deep thought should be given to architectural features, aesthetic proportion and harmony with surroundings.

In addition to the general requirements, short span bridges are called on to meet special conditions. They may have to resist violent external forces such as floods, or the destructive action of the elements where maintenance and protection are slighted because of the relative unimportance of a small structure. Long span bridges are usually of such high clearance and importance that they escape the full consequences of these destructive forces. Short span bridges should then be of more rugged design and be constructed of materials more resistent to the elements with minimum maintenance.

The non-homogeneous or composite nature of concrete gives this material some very important economic advantages. Cement of good quality, can be readily obtained from conveniently located mills all over the civilized world. Aggregates are within easy reach of most projects. Water supply scarcely needs mentioning. Lumber for forms and supports utilizes standard sizes to be found in all localities. Reinforcing steel, a small part of the total tonnage of a structure, is readily available. Maximum utilization of labor has become of prime importance. The labor necessary on a reinforced concrete project is for the most part of unskilled or semi-skilled type which may be drawn from local labor supplies. And since most of the materials are obtained close by, a community receives further labor advantage in the production of these materials.

From the standpoint of physical characteristics, concrete's plasticity, allows the easy molding of details and a simple variation of the proportions of ingredients, its strength and workability for various conditions can be controlled accurately.

Although the principles of structural continuity have been successfully applied to other materials of construction, there is no other in

which it is utilized to more advantage than concrete. Its very physical characteristics invite monolithic construction with consequent simplification, increased rigidity, and opportunities for continuity. It is, therefore, more difficult actually to achieve partially a statically determinate structure than it is to recognize qualities of the material by the design of one frankly statically indeterminate. As this principle is becoming more thoroughly understood and used, designers are forsaking methods in which only statically determinate primary stresses are calculated. In these methods the secondary indeterminate stresses are unscientifically taken care of through low, conservative working stresses and arbitrary assumptions of design.

DETAILS, DESIGN METHODS AND MATERIALS

Just as a chain is no stronger than its weakest link, so is a structure no stronger than its weakest detail. This may apply, not only to the actual physical strength of a structure, but also to its economic strength. It is also axiomatic that, while good structural design may be ruined by poor details, a poor design cannot be saved by good details. In actual construction practice the value, life and efficiency of a structure depend on the proper execution of design and details, and the selection of materials of proper quality. Hence, the problems of design, details and materials must be solved almost simultaneously as each progressive step is taken in the development of more advanced bridge construction.

The early trend of rigid frame and continuous design followed the simpler of the forms of reinforced concrete practice—the haunched slab. Because of weight, it was believed that the maximum economic span limit for concrete elastic frame bridges was about 60 ft. A few attempts were made to utilize ribbed design for longer spans, but these attempts also failed in their objective because of the neglect in adapting and using fundamental concrete details. It has long been known that the ideal concrete structural section for the resistance of flexure is the T-beam, but its use in an inverted position to resist negative bending moments was not introduced until the construction of the Martinez Street bridge in San Antonio, Texas. As a result, engineering opinions on the maximum economic span limits of concrete elastic frames have been revised upward even considering present materials.

Although design methods applied to elastic concrete frames and other continuous structures are still in a transitory stage, progress has been encouraging as evidenced by deep study, extensive research, voluminous writings, and heated discussions. Especially is it interesting that this development of continuity in concrete design is

following a course similar to the development of other familiar bridge building materials.

Since adequate engineering literature has already been published on several different design methods, only general and brief mention will be attempted. Excellent results have been obtained in the design of elastic frames through the use of the Beggs deformeter applied to scale models and through empirical methods evolved by Prof. Hardy Cross who advocates the distribution of fixed end moments in a beam through a series of arbitrary approximations and assumptions. The pioneer of elastic frame bridge design, Arthur G. Hayden, has developed a third method which is an important step in the direction of simplified scientifically accurate design. This latter method is based on the analogy between an elastic frame and an elastic arch in which the arch equations are used in the basic derivations. In its present form, Hayden's method is unsuited to complicated and unusual structures because of laborious calculation involved.

A fourth method of elastic frame and continuous design has been partially perfected by the writer's organization. Our new method is based on a combination of certain of the principles of the "Slope Deflection" and the "Elastic Weights" theories of indeterminate structures, together with some original methods of handling variable moments of inertia. In its final perfection, what we have called the "Elastic Slope" method of continuous design, it is believed that truly scientific design will be greatly simplified.

Conservatism among engineers, of the same type that has retarded the development of details and design methods, has likewise retarded the use of more scientifically controlled strength concrete. Thus ridiculously low working stresses are being used on structures of high quality concrete. Until lack of confidence in high strength concrete is overcome, this material will continue under severe handicaps. As this prejudice is overcome we may expect a consequent lightening of dead loads of structures, enabling increase of spans and slenderness through the medium of design continuity. In some cases continuous and rigid frame concrete bridges have already invaded the field of steel pony trusses and steel girders with considerable saving under the cost of equivalent steel structures.

TEXAS EXAMPLES OF CONTINUOUS AND RIGID FRAME BRIDGES

During the last three years a number of interesting reinforced concrete bridges have been successfully built in Texas. They show the application of our present subject. The bridges in San Antonio and, New Braunfels were designed by the writer's organization under the

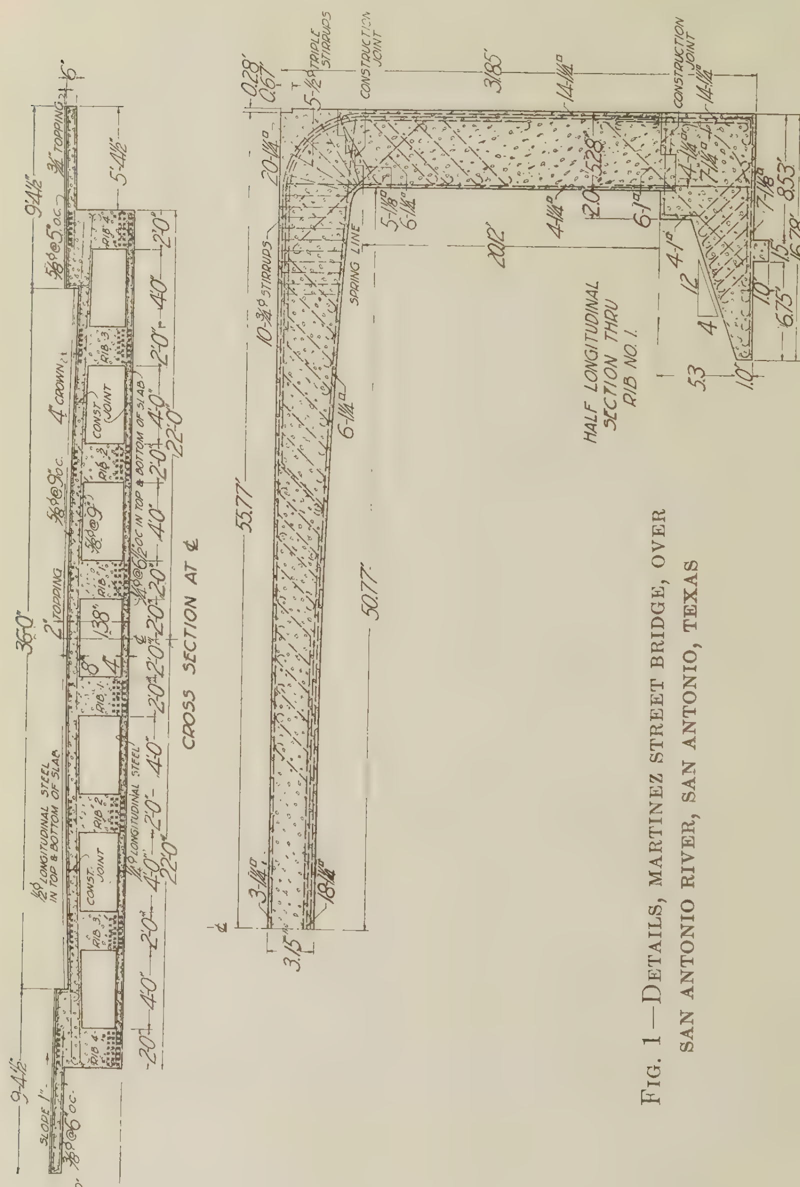

FIG. 1—DETAILS, MARTINEZ STREET BRIDGE, OVER SAN ANTONIO RIVER, SAN ANTONIO, TEXAS

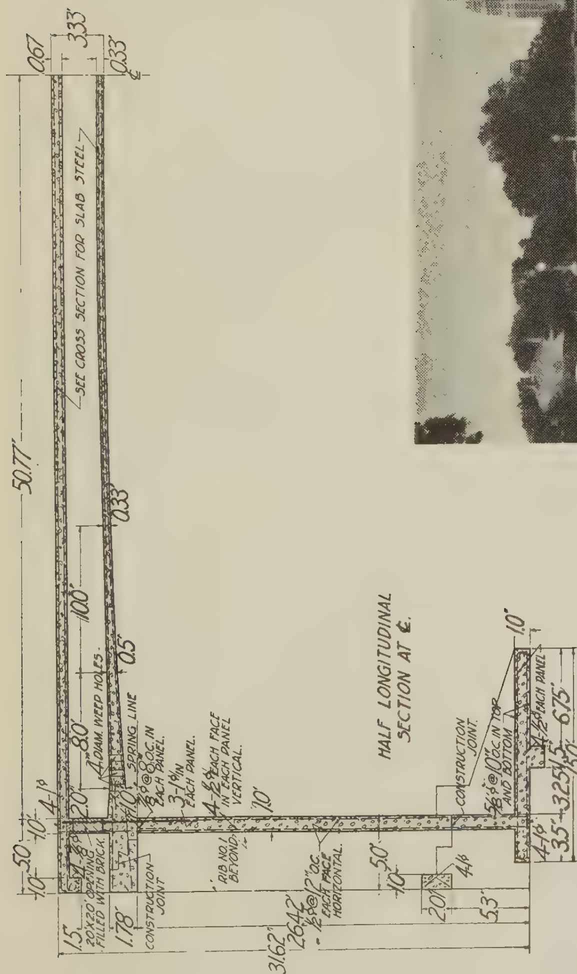

FIG. 2—DETAILS, MARTINEZ STREET BRIDGE

FIG. 3—MARTINEZ STREET BRIDGE AS COMPLETED

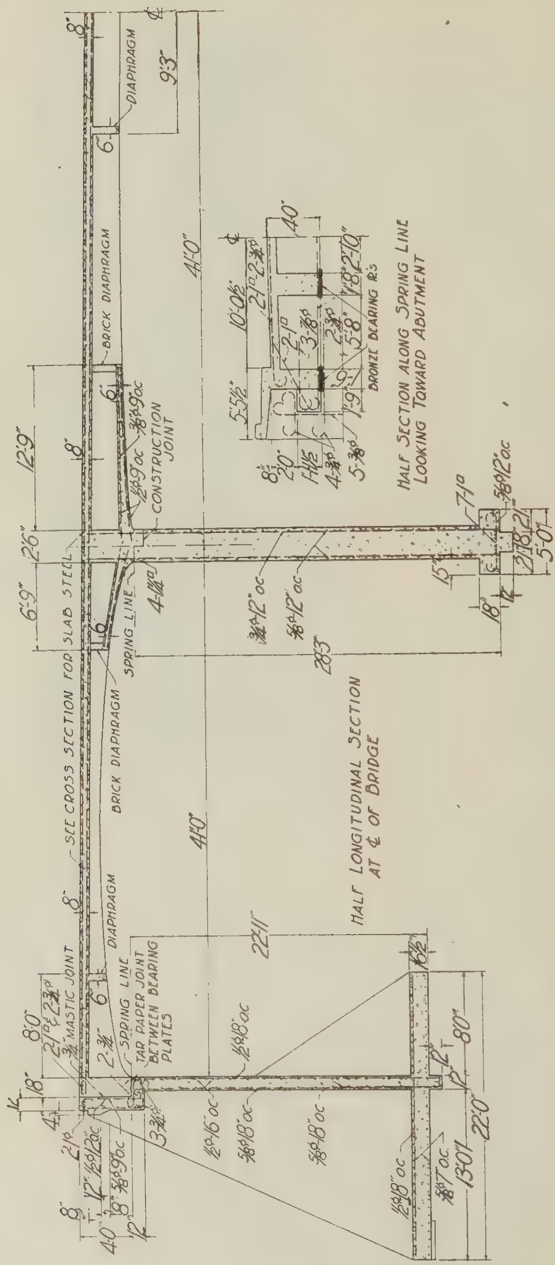

FIG. 4—DETAILS, GARDEN STREET BRIDGE OVER COMAL RIVER,
NEW BRAUNFELS, TEXAS

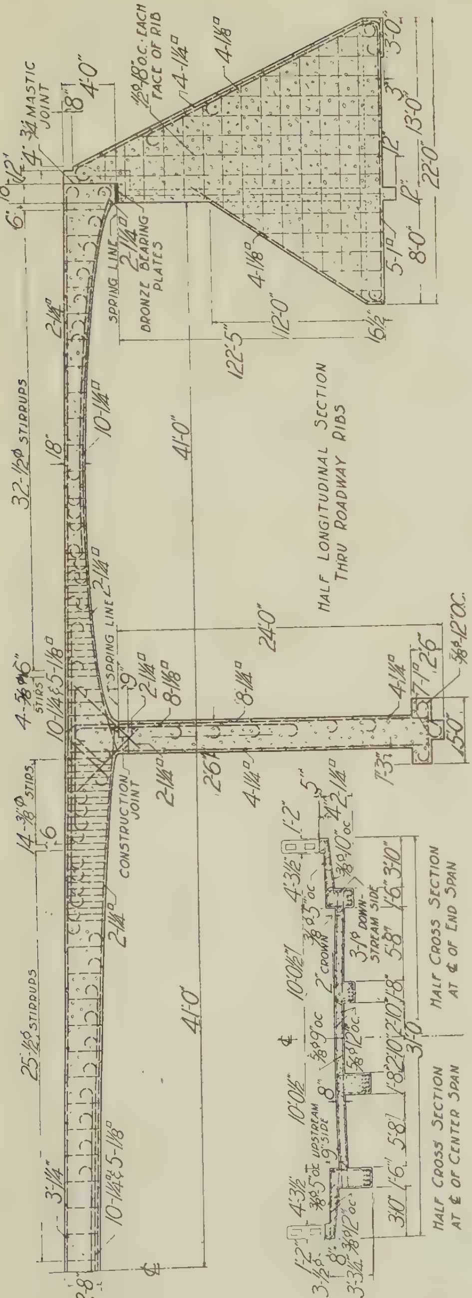

FIG. 5—DETAILS, GARDEN STREET BRIDGE

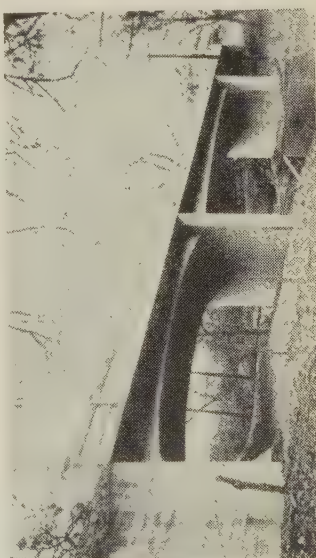

FIG. 6—GARDEN STREET BRIDGE AS COMPLETED

direction of W. E. Joor. The bridges in Austin were designed by H. R. F. Helland, consulting engineer. The bridges in Houston were designed by J. G. McKenzie, City bridge engineer. Since all of them have been adequately described in previous engineering literature, brief mention only will be made here.

All of the bridges, except those at Austin, utilize high strength concrete designed and controlled by the water-cement ratio. Controlled concrete was accompanied by the use of the high design stresses allowed by the Specifications of The Joint Committee. The bridges at Austin followed more conservative and older low working stresses with consequent sacrifice of economy and proportion. The structures were all designed for standard live loadings with proper allowance for impact in accordance with the approved specifications now in force in Texas. In general, they are excellent examples of the economic and structural advantages obtained through an intelligent use of advanced indeterminate design, suitable details and improved concrete construction technique.

The Martinez Street bridge, crossing the San Antonio river, in the City of San Antonio, is of the single span, ribbed type rigid frame bridge having a 10 deg. skew, a clear span of 101 ft. 6½ in., a roadway width of 36 ft., and two 8-ft. sidewalks. The total depth of the structure is 38 in. at the center and 65 in. at the face of the abutment. Two distinct innovations in details were introduced by the soffit slabs and cantilevered foundations which were used to meet special bending moment conditions. From available records it appears that this bridge is the longest single span concrete rigid frame structure in the United States.

The Garden Street bridge over the Comal river in New Braunfels, Tex., is composed of a center span of 80 ft. clear and two side spans of 40 ft. clear which, including piers, give a total length of 164 ft. on a 20 deg. skew. The roadway is 20 ft. wide, flanked by two sidewalks, each 4 ft. 4 in. wide. The design utilizes a combination of the principles of horizontal continuity of the three spans and rigid frame continuity with the piers. Expansion details provided at each abutment, soffit slabs of improved type, as well as simplification of reinforcing, provided interesting development of details.

The McKee Street bridge in Houston, Tex., utilizes the principle of horizontal continuity only without rigid connection to the piers. It is of three span, partial through-girder type with a center span 120 ft. and two end spans of 85 ft. each. It is built on a 10-ft. skew with a clear roadway width of 38 ft. 4 in. between girders, flanked by two 5-ft. sidewalks. Depth of the girders varies in accordance with the

moment diagram, with the bottom level. The girders have a maximum depth over the intermediate supports of 14 ft. Exposed portions of the concrete, visible from the roadway, are painted with aluminum paint. It is believed that the center span is the longest reinforced concrete bridge girder in the United States.

Another bridge in Houston, Texas, the Seventy-Fifth Street Bayou crossing, is of the haunched slab type continuous over five spans without rigid connection to piers. The center span is 60 ft. flanked by two other spans of 50 ft. with two outside spans of 22 ft. 6 in. each. The roadway is 38 ft. wide flanked by two sidewalks each 5 ft. 6 in. Two rocker piers assist in taking care of expansion and contraction of the structure.

The only rigid frame bridge as yet built in Houston, Tex., is the Seventy-Fifth Street underpass, of the two span type. It is of the barrel slab type, having two clear spans of 21 ft. and a headroom clearance of 14 ft. above the roadway. The structure carries a single track railway supported on ballast and has a walkway about 3 ft. 8 in. wide on each side. Since this structure is designed for Cooper's E-65 loading plus 100 per cent impact, it is believed to be carrying the heaviest live load yet placed on a rigid frame bridge of reinforced concrete.

The bridge at Alameda Street in Houston is continuous over three spans with a center span of 60 ft. center to center of piers and two side spans of 40 ft. each. The roadway is supported on continuous deck girders spaced about 10 ft. on centers and in turn support floor beams about 10 ft. on centers. The effect is to lay the roadway slab out in square panels. The Telephone Road bridge in Houston is similar.

Two bridges over Waller Creek in Austin are also of the rigid frame type. These are at Thirty-Second street and Thirty-Fourth street and are of practically identical dimensions, differing only in exterior decorative detail. They are of 39 ft. clear span, carrying a 30-ft. roadway and two sidewalks each 5 ft. 6 in. wide. They are of the deck rib type with the soffits having a deep graceful curve which gives the appearance of arches. A hard limestone foundation enabled the utilization of details which closely approximated pin connected conditions at the base of the vertical ribs.

THE FUTURE OF RIGID FRAME AND CONTINUOUS CONCRETE DESIGN

The future of short span bridge design is largely dependent on three factors: first, an encouragement of independent research effort in design; second, a development and improvement of construction materials; third, an intelligent and economical use of materials and

methods. If independent engineers are assisted and permitted to develop their ingenuity, revolutionary improvements in design and construction details will undoubtedly be developed. The further development of materials and construction technique will surely follow.

With statically determinate design already so well understood and developed, there is very little promise of developments along this line. We must look to the great new field of statically indeterminate design for the future progress of short span bridge design. The progress to date, is but an indication of what can be accomplished. The trend of short span bridge design points surely to the concrete rigid frame and continuous bridges as the dominant types of the future. With many other advantages, such bridges would be valuable in economy alone.

Buildings also can profit from the developments of concrete elastic and continuous frames. The writer's organization recently completed the design and construction of a large church building in San Antonio, in which certain difficult structural problems were solved through the use of concrete rigid frames. In buildings of the industrial type requiring long spans, the rigid concrete frame also has a wonderful field of usefulness—especially for mill buildings and airplane hangars.

The development and improvement of construction materials may only be surmised as this field is one of the chemist and physicist. Development should reasonably follow the course of improved quality, easier application to construction methods, higher strength and lower weight. It is not unreasonable to suppose that light weight, high strength alloys will be developed for use as reinforcing in place of our present grades of steel. It is also not unreasonable to expect future development of light weight concrete aggregates and higher strength cement. Improvement in the use of materials will increase the dependability and uniformity of concrete so that higher working stresses can be used. In general, therefore, the trend is toward the reduction of the dead loads of structures built of concrete, and as this is achieved we may expect gradual extension of the economic limits of the length of concrete bridge spans.

Before the surmised developments can become facts inertia among engineers must be overcome. A great French engineer, E. Freyssinet, already is considering the possibility of concretes under present conditions in which ultimate strengths will reach 16,000 p. s. i.

For such discussion of this paper as may develop, readers are referred to the JOURNAL *for September, 1934 (Proceedings, Vol. 31). Discussions should be available to the Secretary by June 1, 1934.*

Proper Methods of Design and Construction of Concrete Structures to Prevent Damage From Volumetric Changes of the Concrete*

Report of Committee 102—Volume Changes in Concrete

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MEMBER AMERICAN CONCRETE INSTITUTE

INTRODUCTION

In a previous report by the committee‡ Raymond E. Davis submitted an excellent resumé of the results of investigations having a bearing upon the general subject of volume changes of mortars and concrete, including those concerned with (a) the effect of moisture conditions and (b) the effect of variations in temperature.

The purpose of the present report is (1) to summarize briefly the information presented in the earlier report; (2) to outline research needed in connection with various phases of the general problem to fill the gaps in our knowledge of the effect of certain factors upon volume changes which take place with time and variations in temperature; and (3) to indicate how the damage of structures due to volume changes in the concrete may be at least partially prevented by proper choice of the materials used and the design of the mix, by correct design of the structure, and by proper construction methods.

Ordinary volume changes in concrete structures may be reduced by judicious selection of the concrete materials; using a cement known to produce small volume changes, by the use of limestone or quartz aggregate in preference to sandstone or gravel containing many sandstone particles, and unless a high degree of workability is desired, by the selection of a fine aggregate having few particles passing the No. 50 sieve which are flat or disc-like in shape and which become slippery when wet. The proper design of the concrete mix is also essential if small volume changes are desired because (1) more cement than

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necessary for strength, stiffness, workability, imperviousness or durability, will give greater shrinkage or expansion than a leaner mix, (2) use of more fine aggregate than is required for necessary workability will cause greater volume changes than would result from a more coarsely graded combined aggregate, and (3) the shrinkage of concrete exposed to drying in the air may be reduced by using as low a water-cement ratio as is consistent with a workability adequate for proper placement.

The structure itself may be designed so that the resulting shrinkage stresses will not be excessive, and it may be so built that stresses which might otherwise develop will not be present to cause cracking, thus enhancing the structure's appearance and its service. In a long structure, provision should be made for expansion or contraction joints at suitable intervals to relieve stresses certain to result from deformations caused by variations in moisture and temperature conditions. Flexible columns in the first story will relieve shrinkage stresses introduced by the fixedness of the foundations. Additional steel reinforcement in reduced sections of the structure subject to high stress due to volume changes, will prevent large cracks. In construction, the use of absorptive forms will aid in reducing volume changes. The loosening of wall forms at window-openings about 15 hours after placing the concrete will aid materially in preventing cracks so often seen adjacent to the lower corners of windows. In massive concrete structures, cracking caused by the contraction while cooling from the high temperatures developed by the setting of the cement, may be partially prevented by (1) building the mass in units with narrow gaps in between so as to permit natural cooling of the units first constructed before the intervening narrow gaps are filled in, (2) the use of precast units, (3) the placement of the concrete at low temperatures, and (4) the cooling of the mass by mechanical refrigeration while heat is being liberated by the setting cement.

SUMMARY OF THE FACTORS INFLUENCING VOLUME CHANGES

Tests have shown many factors influencing the contraction of concrete with loss of moisture and its expansion with gain of moisture. These changes are approximately 100 per cent greater for a 1:1:2 mix than for a 1:3:6 mix, as the average contraction per 100 ft. after one year in dry air amounts to about 1.1 in. for the rich mix and 0.6 in. for the lean mix. The cement used may also exert a marked influence, changes occurring in some neat cements being more than double those occurring in other cements of the same general fineness although such large variations are not common in modern cements. The expansion

of cement upon immersion is particularly increased by the presence of free lime or magnesia. The higher the tricalcium aluminate content, the greater the expansion under moist conditions. The higher the dicalcium silicate content, the less the expansion under water but the greater the contraction in air. An appreciable increase in loss on ignition materially increases the contraction in air. Although fineness of cement has a small influence upon volume changes at the early ages, the effect at later ages is negligible.

The heat of hydration of a cement used in mass construction exerts a marked influence upon volume changes resulting from temperatures developed within the mass, and the chemical composition of cement has an important effect upon both the rate and total amount of the heat of hydration during any period of time. Tricalcium aluminate liberates more heat per unit of weight than does any other major compound, and this liberation very largely occurs during the early stage of the hardening process. The higher the tricalcium silicate content the more rapid the development of heat and the greater the temperature rise.

The volume change tendencies of the aggregate have a direct bearing upon the behavior of the concrete so that, in general, the relative shrinkage of concrete made of gravel (containing many sandstone particles), sandstone, trap rock, slag, granite, quartz or limestone aggregate decreases in the order named. Variations as great as 100 per cent may be expected between gravel and limestone concretes.

If the fine particles of the aggregate are smooth and disc-like and tend to become slippery when wet instead of being angular and feeling rough and gritty when rubbed between the fingers, an increase in the fineness of the aggregate will result in greater volume changes. In general, however, the gradation of the aggregate has a much less effect upon volume changes than its mineral character.

The effect of an admixture upon the volume changes of concrete appears to depend more upon the mineral and granular character of the added material than upon the mere fact that finely divided particles have been added to the mass. Some admixtures, such as hydrated lime, appear to have very little effect whereas others, such as diatomaceous earth, produce a definite increase in the shrinkage.

For rich mixes, an increase in the water-cement ratio may produce 20 per cent more shrinkage than do dry mixes but the difference is not great for the lean mixes.

The length of the initial period of water curing and the method of curing at normal temperatures have no appreciable effect upon the

ultimate air shrinkage, although careful curing may aid in the development of strength so as to make the concrete better able to withstand shrinkage stresses without cracking. However, curing for 28 days under conditions simulating those in mass concrete produces volume changes about half those for normal curing temperatures.

The rate of volume change depends upon the opportunity for loss or gain of moisture by the structure, thin sections changing at a more rapid rate and reaching equilibrium more quickly than more massive sections.

Tests show that by the judicious use of steel reinforcement, shrinkage of the concrete may be reduced and if the structure is not too long between joints, the development of large shrinkage cracks may be prevented. It has also been shown that by the use of absorbent molds, the shrinkage may be reduced by from 15 to 50 per cent for ordinary and rich mixes (by reducing the final water-cement ratio of the mix).

The thermal coefficient of expansion is affected more by the type of aggregate than any other factor. In general, the coefficients for concrete made of quartz, sandstone, gravel, granite, basalt or limestone aggregate decrease in the order named, the coefficient of 0.000,0038 per 1°F. for the limestone concrete being about 60 per cent of the 0.000,0066 value for the quartz concrete. Variations as large as 30 per cent due to the brand of cement have also been observed in the coefficient for neat cement and the richer the mix the greater the value of the coefficient, a difference of about 10 per cent existing between a 1:3 and 1:6 mix. The coefficient for wet concrete may be as much as 5 per cent higher than for the same concrete when air dry, but age appears to have very little effect.

FURTHER RESEARCH REQUIRED

Even though many data have been obtained from investigations of volume changes in concrete, there are phases of the problem which may well be the object of further research to fill in the gaps in our present knowledge of the general subject.

Although there appears to be a very appreciable difference between the volume changes produced by variations in moisture and thermal conditions for test bars made of different cements, little work in addition to that now in progress at the University of California in connection with the Boulder Dam Cement Investigation has been done to correlate the effect of the chemical composition of the cement and the manufacturing processes employed with the expansion or contraction resulting from its use in a mortar or concrete mixture. Research to

determine this relationship may lead to the development of a cement which when used in actual structures will result in somewhat smaller volume changes than those commonly experienced.

Parallelling the study of the effect of the chemical composition of the cement, an investigation might well be undertaken to determine the effect of the gradation of the cement particles, particularly the gradation of those passing the No. 200 screen, upon the volume changes produced. Some investigators have reported test results for certain finely ground cements which appear a bit odd in the light of our more common knowledge of volume changes.

Admixtures to concrete are being given a good deal of attention, and it appears that they may receive more in the future. We know relatively little concerning their action in concrete mixtures, particularly with reference to the volume changes resulting from their use in comparison with those occurring in similar concretes without admixtures, although some investigators report that the admixtures tested by them cause greater volume changes. The physical characteristics of the individual particles of the many admixtures on the market probably differ materially, some consisting of flat, disc-like particles, while others are spherical or polygonal, and it may well be that these variations, along with those resulting from dissimilar behavior in the presence of moisture, are sufficient to produce significant differences in the volume changes resulting from their use in concrete mixtures. Research may disclose that certain admixtures are desirable because of either greater workability, yield or strength, combined with a decrease in volume changes resulting from variations in moisture or temperature conditions.

It has been shown that expansion and contraction of mortars and concretes due both to moisture and temperature, are appreciably affected by the type of aggregate, the values for sandstone concrete, for example, being about twice or even considerably more than twice the corresponding values for limestone concrete. All materials used as aggregates have not been subject to investigation, so that there is opportunity for further research along this line.

In addition to differences caused by the type of aggregate, it has also been shown that the gradation of any particular fine aggregate, and the characteristics of the particles themselves—whether polygonal and gritty, or flat, disc-like and slippery when wet—has an appreciable influence upon expansion and contraction, but the available information on this factor is still incomplete.

Very little is known concerning the effect of size and shape of the concrete mass under observation upon the volume changes which occur

as the investigators who have studied the shrinkage and expansion of concrete have usually carried on their tests using a single size and shape of test specimen. As stated previously, it is probable that the moisture content will vary more from point to point in a large mass than in a small one and that these variations will tend to produce different volume changes at the faces of the mass than at its center. Thus it appears that there must always be set up, as volume changes take place, stresses, part of which are tensile and part of which are compressive in character. It might be expected that these stresses would produce a plastic yielding or flow of the concrete permanently elongating the tensile fibres and shortening the compressive fibres so that in a large, thick mass, an ultimate shrinkage might be expected considerably less than in a small thin mass. This factor provides a problem for further research and is one which should yield results of considerable value to the engineer concerned with massive concrete structures.

PROPER SELECTION OF MATERIALS AND DESIGN OF MIX TO REDUCE VOLUME CHANGES

All investigations having to do with volume changes of concrete mixtures have shown a marked difference between concretes depending upon a number of factors which have been discussed and their effects summarized. From a study of this summary it is apparent that by careful attention to the selection of the materials of which the concrete is to be made and by diligent application of certain fundamentals pertaining to the proper design of concrete mixtures, the resulting concrete will undergo much smaller volume changes due to loss or gain of moisture and also due to increase or decrease of temperature.

In the proper selection of concrete materials, the most important item is the aggregate. Volume-change studies have shown that with all other factors constant, certain types of aggregate produce nearly double the volume changes resulting from the use of other aggregates. This may possibly account for the fact that for two structures similarly located and of similar design but constructed of different aggregates, one may be in excellent condition, whereas the other may be cracked severely. Such a situation could easily develop if the concrete of the first structure had limestone or granite aggregate and the second structure sandstone aggregate, or of gravel containing many sandstone particles, as the former structure would experience much smaller volume changes than would the latter.

Table 1, which presents the effect of type of aggregate upon volume changes, being based upon tests of 1:2:3 mixes of concretes varying

from one another only in the type of aggregate, clearly shows the favorable characteristics of limestone concrete when compared with gravel concrete. In this table, the second and third columns give the percentage contraction in air and the percentage expansion in water between the ages of 1 day and 3 months; the fourth column gives the total range of contraction plus expansion for the same period, the values being the summation of those appearing on columns 2 and 3. The fifth column gives the coefficients of thermal expansion of similar concrete bars stored in dry air and tested at temperatures varying from 15° to 130°F.

A study of this table shows that limestone concrete is far superior to gravel or sandstone concrete as regards volume changes due to variations in both moisture and temperature. Changes occurring in the limestone concrete are but little more than half those occurring in the gravel concrete. From the data of Table 1, it can be shown that the air drying of concrete for 3 months produces about the same volume change as a drop in temperature of 95°F. and that the total range of contraction in air for one specimen plus expansion due to water immersion for another is practically equal to that resulting from a temperature change of 110°F. It may be seen, therefore, that for many structures, moisture and temperature changes are of about equal importance in determining the volume changes and the resultant state of stress of the structure.

TABLE 1—EFFECT OF TYPE OF AGGREGATE UPON VOLUME CHANGES OF 1:2:3 CONCRETE

Kind of Aggregate	Percentage Change in Three Months			Coefficient of Thermal Expansion per 1°F. Age 3 Months
	Contraction in Air	Expansion in Water	Range of Contraction Plus Expansion	
(1)	(2)	(3)	(4)	(5)
Gravel	0.079	0.007	0.086	0.0000060
Sandstone	0.075	0.006	0.081	0.0000065
Granite	0.037	0.013	0.050	0.0000053
Quartz	0.036	0.009	0.045	0.0000066
Limestone	0.039	0.005	0.044	0.0000038
Average	0.053	0.008	0.061	0.0000056

Considering a climate where temperature variations are of equal importance with moisture in producing volume changes, it is evident from a study of this table that limestone and granite possess certain advantages over gravel containing many sandstone particles, or to crushed sandstone as aggregate for concrete mixtures. This should not be construed to mean that all gravel aggregates are inferior to all limestones and granites in regard to the development of large undesirable

volume changes, since obviously behavior in this respect would depend greatly upon the character of the gravel.

Not only is the type of aggregate of importance but its gradation also exerts some influence upon the resulting volume changes. It has been demonstrated that when the combined aggregate contains excessive silt or material passing the No. 50 sieve which, instead of being gritty is slippery when wet and imparts considerable smoothness to the mix, the resulting concrete will undergo greater volume changes than would be the case if such material were not present in the mix. Although the tentative standards for concrete aggregates of the American Society for Testing Materials permit the material passing the ¼-in. screen to contain as much as 5 per cent of material which will pass the No. 100 screen and 30 per cent to pass the No. 50 screen, such a large proportion of fine material should not be permitted as a rule, particularly if the fine particles are of the flat, disc-like type in contrast to those which when rubbed between the fingers feel rough and gritty.

This specification may be satisfactory if workability alone is to be considered—although this is usually not the case—but these percentages are higher than is desirable if small volume changes are essential. For small volume changes, the percentage of these fine particles should be reduced to a minimum consistent with fair workability and lack of bleeding of mix. While proper workability is a necessary property of all concrete mixtures, and within certain limits the addition of fine particles makes for smoothness of the mix, it must be understood that what is a mix of proper workability in one case may be one with an excess of fine material in another case. It should be noted also that, for some structures, as those subjected to water pressure and for which the concrete must be impermeable, some fine inert material is essential to fill spaces between the larger particles. Always, however, care should be taken in grading the aggregate for a particular structure to see that an excess of material passing the No. 50 sieve is not included. For mass concrete, not over 15 per cent of such fine material is necessary; for ordinary construction, about 20 per cent by weight of the fine aggregate, is all that is necessary to pass the No. 50 sieve to give fair workability to the mix; and for thin hydraulic structures or for special pours which require a very workable mix, about 25 per cent of the fine aggregate passing the No. 50 screen should prove sufficient.

The use of admixtures is related very closely to these considerations since most commercially available admixtures are either (1) in a finely powdered form so that their use increases the material passing

the No. 50 screen or (2) they are liquids which tend to develop colloidal substances within the mass. The concrete made with either of these two types of admixtures will usually suffer greater shrinkage due to the contraction of the colloid produced by the liquid admixture, or to the excess water required to moisten the powdered admixture and the later loss of this moisture.

If the concrete aggregate is naturally deficient in fine particles or has lost them by excessive washing, the addition of some admixture may be desirable to promote either greater workability or a less permeable mass. In some cases, these desirable properties may be obtained at less expense and with equal effectiveness by less thorough washing of the aggregates.

Different portland cements, when tested in neat cement bars, are known to produce variations as great as 100 per cent in the amount of shrinkage produced upon air drying and in the expansion resulting from water soaking. Also, length changes due to temperature variations have been shown to vary 30 per cent. It therefore behooves the engineer to investigate the volume changes resulting from the use of the various local cements to see if they differ materially in this respect. If a laboratory investigation of the cements is not possible, it may suffice to examine structures made from the cement which it is proposed to use, to see if there is evidence of unusual cracking caused by excessive volume change.

The difference in action of the various cements is due primarily to its chemical composition, although other factors such as manner of grinding the raw materials and of burning, heat treating and seasoning the clinker may be of equal importance. The fineness of grinding has been shown to have little ultimate effect upon volume changes.

As for the effect of chemical composition of the cement, it has been shown that cements high in tricalcium aluminate, tricalcium silicate, or magnesia show greater expansion under water but slightly less contraction in dry air. On the other hand, cements high in dicalcium silicate or having large losses on ignition due to continued storage in a moist atmosphere show considerably greater contractions than do normal cements when in dry air.

After the concrete materials have been selected with a view to the development of small volume changes, comes the proper design of the mix, which in this discussion means the determination of its richness and water-cement ratio.

The concrete should be made as lean as is possible for the development of adequate strength, stiffness, imperviousness, durability and

other essential properties because any excess cement increases volume changes. Moisture variations may produce about double the volume change in a 1:1:2 concrete mix that would occur in a 1:3:6 mix, and temperature variations may produce about 10 per cent greater changes in a 1:3 mix than in one of 1:6 proportions.

While the richer mixes should be stronger and better able to withstand stress caused by volume changes, it appears that the richer mixes suffer greater cracking. It is thus better practice to use the lowest cement ratio consistant with necessary properties, rather than to develop excessive strength in an attempt to resist cracking from shrinkage stresses. Excessively rich mixes are usually forestalled by the cement cost, so that in general we do not need to be concerned much about the excessive shrinkage of the richer mixes. However, in special instances, such as in cement stuccos or on jobs involving but little concrete, the richness may be increased to an undesirable degree without adding materially to the total cost of the structure. Experience with cement stuccos appears to indicate that the base or scratch coats should not be richer than 1:4 and that the finish coat should not be richer than 1:3 for the best results.

The water-cement ratio should receive careful attention in the design of the mix, not because the difference in contraction of a wet mix is so much greater than for a dry mix (20 per cent for rich and about 5 per cent for lean mixes) but because where the richness of the mix is fixed by specification and sufficient water is added to give a suitable workability, it is so easy to use more water than is necessary and produce greater shrinkage than would be experienced were good judgment used in determining essential workability. If a definite water-cement ratio is specified for the job, the contractor will ordinarily use a mix as stiff as will permit satisfactory placement, because any increase in workability by the addition of water requires a similar increase in the amount of cement. This, of course, tends to keep the water content to a minimum, so that the use of the water-cement ratio principle in the design of concrete mixtures is a satisfactory method of preventing large volume changes due to an excess of mixing water.

PROPER DESIGN PRINCIPLES AND CONSTRUCTION METHODS TO AVOID DAMAGE FROM VOLUME CHANGES

With the greatest care in securing the most satisfactory concrete materials and the best mix design for the structure, some shrinkage of the concrete upon drying out will surely occur. It is thus essential that the structure be so designed that the shrinkage will not cause damage by developing unsightly cracks and that volume changes due to tem-

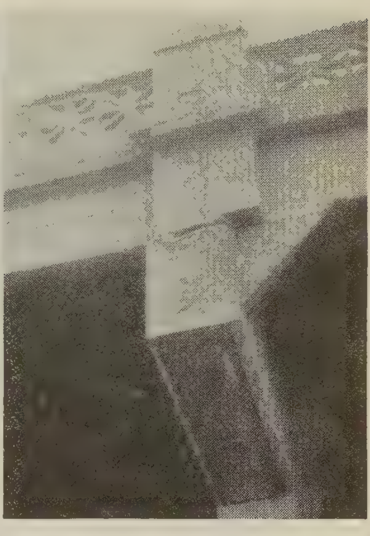



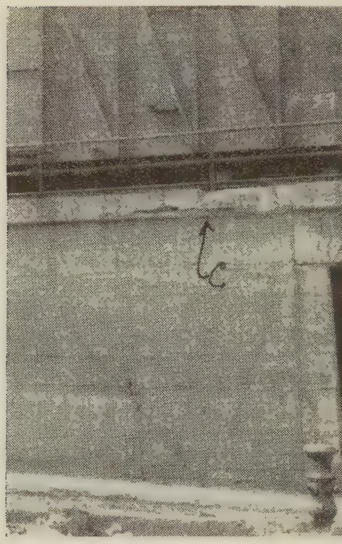
FIG. 1—PILE CAP—DISRUPTION RESULTING FROM ADHERENCE OF DECK BEAM WHERE FREE MOTION HAD BEEN INTENDED

FIG. 2—VIADUCT PILASTER. CRACKED FOR LACK OF FREEDOM AT THE EXPANSION JOINT

perature be provided for. It may be stated as a general rule, that every design should be such that expansion and contraction from variations in moisture or temperature may occur without injury to the structure. That this rule has not prevailed may be shown by a casual survey of the concrete structures in almost any locality.

The disruption of the pile cap girder shown in Fig. 1 (part of a bridge in Florida) is an illustration of what may occur due to inadequate provision for contraction of the longitudinal deck beams of a bridge. Here the provisions for movement of the longitudinal deck beams with respect to the pile cap girder were inadequate, as the deck beams adhered to the concrete of the girder and caused extensive spalling. This could have been avoided had more positive freedom of action been established. The use of tar paper to break the bond, and even the use of bronze seats, have in general proved to be ineffective in providing the necessary freedom.

Damage from a somewhat similar cause is shown in Fig. 2, a viaduct span in Texas, where the pilaster cracked due to a lack of freedom at the expansion joint in the bent shown. Evidently, the pilaster seized on to the supposedly free end of the beam without haunches, as effectively as to the fixed beam having the haunch.



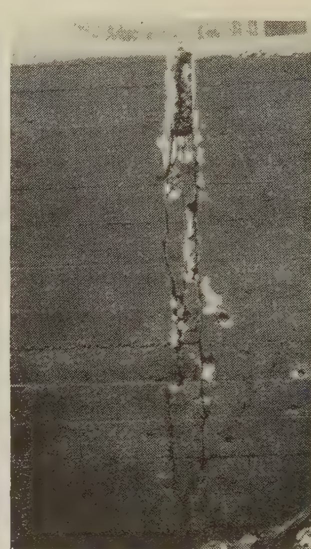


FIG. 3—RETAINING WALL COPING FAILS BY COMPRESSION DUE TO INSUFFICIENT PROVISION FOR EXPANSION BETWEEN GIRDER AND WALL

FIG. 4—RETAINING WALL JOINT FAILS DUE TO LACK OF FREEDOM BETWEEN TONGUE AND GROOVE AND TO INADEQUATE REINFORCEMENT OF GROOVE

Hydraulic structures are particularly subject to damage by cracking due to volume changes. Cracks in a building or like structure are unsightly but do not seriously endanger it, nor do they usually make it less serviceable, whereas in reservoirs and similar structures the cracking of the walls is a matter of grave concern. A striking example of the damage resulting from lack of proper consideration of the shrinkage of concrete with age, is the severe cracking of a water filtration plant in Texas, even though at least one face of each wall is more or less continuously wet. This structure is about 60 by 160 ft. with no provision whatever for shrinkage of the concrete. Even though the structure is reinforced to care for the normal load stresses coming upon it, the steel was inadequate to prevent cracks, some of which were as wide as 3/16 in. Experience has shown that it is not only economically impracticable but that it is also impossible to prevent cracking by the use of extra steel alone, the only practical solution being the introduction of effective contraction joints at intervals which will vary with climatic and other conditions. For hydraulic structures in temperate climates, where the humidity and range in temperature is moderate, the spacing of joints probably should not exceed about 40 ft., whereas in districts

of low humidity and large range in temperature, the spacing should be somewhat less.

The necessity of providing ample clearance for expansion due to high temperatures is shown by Fig. 3, a retaining wall in Chicago, in which it may be observed that expansions of the girder and of the wall completely closed the joint which had been provided between the girder and the wall, subjecting the coping to greater compressive stresses than it could stand and producing the failure at the point "C."

Fig. 4 shows another damaged retaining wall in Chicago. The two parts of this wall meet with a tongue and groove joint, the tongue being on the section to the right. The sides of the groove apparently were not reinforced, so that as contraction occurred, the adherence of the tongue to the groove ruptured the latter at its connection with the wall. This might have been prevented by adequate reinforcement of the groove and by provision for freer movement at joint. The cracking shown in Fig. 4 may have been caused partially by the freezing of moisture between the tongue and the groove, in which case neither of the two precautions cited would have been completely effective.

What occurs in long retaining walls without expansion joints may be shown by reference to a high, heavily reinforced concrete retaining wall several thousand feet long and without expansion joints which was built a decade ago along a railroad yard on Staten Island. During construction of the wall it was stated in defense of the design that expansion joints had not been provided as they were not needed, the quantity of steel being relied upon to prevent cracking. About six months later, on an inspection of 1000 ft. of this wall, there were counted 45 cracks extending from the base to the top, and 17 extending part way, making an average of about one crack in every 16 ft. Through 8 of these cracks water was issuing at various heights.

The steel in the structure generally has little effect in preventing small cracks in the concrete. In building the retaining walls for the Pennsylvania Railroad terminal, in New York City, it was found impossible to make the sections longer than 25 ft. without cracks. About 50 per cent of the reinforced concrete floor beams, which carry the concourse floor of vault light construction in the Pennsylvania Railroad Station, New York City, show cracks where they join the columns. These were built as continuous beams over great distances, and the contraction of the concrete while drying caused the cracks.

Numerous investigations have been made of the expansion and contraction of concrete highways to ascertain the most effective solution of this ever-vexing problem. Unreinforced highways without con-

traction joints will usually develop transverse cracks about every 25 ft., the actual spacing varying somewhat with a number of factors. In studies of experimental concrete highways, it was observed that with a high percentage of continuous steel, relatively fine, closely spaced cracks may be looked for; with a low percentage, breaks in the steel may be expected to permit wider cracks to form at considerable intervals. It has been stated that there is possible danger in the use of too high a percentage of longitudinal steel, for under such conditions numerous fine transverse cracks will develop and there is possibility that the narrow transverse beams thus formed will crack under traffic.

The best practice in dealing with this problem appears to consist of three distinct steps: (1) the construction of definite transverse joints at regular intervals of, say, 25 ft., (2) the use of longitudinal reinforcement along the two sides of the slab and the use of transverse reinforcement on each side of the joint to prevent cracking of the concrete due to the impact of concentrated wheel loads and to tie each small slab into a stronger unit, and (3) the use of steel dowels across the joint, these being placed so as to slip freely on one side of the joint, and tending to transmit shearing stresses from one side of the joint to the other.

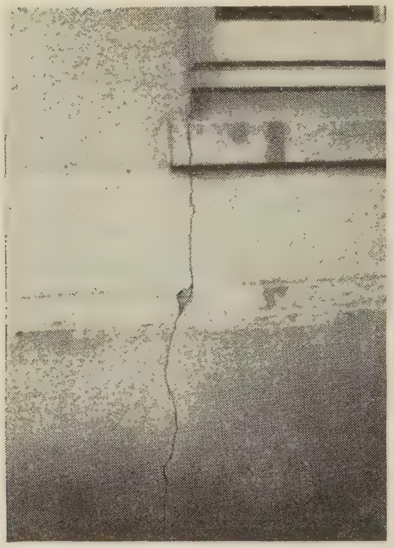
In the design of compression members of reinforced concrete, cognizance should be taken of the effect of shrinkage of the concrete when in dry air upon the stresses in both the concrete and the steel, because it has been shown that when shrinkage occurs in a reinforced, unrestrained structure, compressive stresses are developed in the steel and tensile stresses are set up within the concrete. Without any load whatsoever upon the structure, compressive stresses as high as 19,000 p. s. i. have been observed in the steel when in an amount equal to that oftentimes used in small columns. Even in more heavily reinforced, loaded columns, the shrinkage upon drying, combined with the added compressive stress in the steel caused by the plastic flow of the concrete when under sustained load has produced compressive stresses in the steel equal to the yield point strength of the material, even though the stress as computed according to the commonly used theory for reinforced concrete was only about one-third that value. This leads one to believe that it might be on the side of good judgment to modify our accepted theories as used in the design of reinforced concrete compression members and that consideration be given to the stress developed in the steel by the shrinkage and flow of the concrete. The tests in progress at Lehigh University, the University of Illinois and the University of California to determine the effect of shrinkage and flow of concrete should yield information of value in the development

of more adequate formulas for determining the resistance of reinforced concrete compression members.

In reinforced concrete structures restrained against longitudinal movement due to the length of the structure, the fixity of its footings, or its connection with some immovable object, the shrinkage of the concrete produces longitudinal tensile stresses in the mass. If the structure is long enough so that an appreciable shrinkage tends to develop, the stress will probably exceed the tensile strength of the concrete and cause a small crack to appear. The development of this crack and the accompanying release of stress in the concrete, immediately results in a tensile stress in the longitudinal reinforcement which is oftentimes so high that it exceeds the yield point strength of the steel. The yielding of the steel at this high stress and the further shrinkage of the concrete causes the crack to open wider so that it becomes noticeable. These cracks may be seen usually at points where the concrete is of reduced section and where therefore higher stresses are developed in both the concrete and the steel. The corners of window openings are particularly likely places for cracks due to shrinkage stresses for three reasons: (1) the concrete section is reduced as noted above, (2) a high concentration of stress always occurs at the sharp corners of any opening of members subjected to loading, and (3) during the first 24 hours, a differential settlement occurs between the concrete below the window opening and that at its side.

Fig. 5 shows the type of crack resulting from the first two of these factors in a California building. In particular, note that the crack is of approximately equal width throughout its length. Such cracks usually occur in the lower story of a structure, as at higher elevations the freedom of the end walls tends to relieve the stresses which would result otherwise.

The California building shown in Fig. 6 is another illustration of the cracking which occurs at sections through the wall openings. The crack "C" over the central doorway not only occurs at the mid-length of the building, where cracks are most likely to appear, but it also occurs at the minimum effective section anywhere adjacent to the central portion of the structure, the ruptured wall being smaller than the combined sections above and below the large windows. Although it is practically impossible to eliminate all cracking from a structure of considerable length, it is usually possible to render such cracks less noticeable by causing several very fine cracks to develop instead of one large one. This can usually be done by using more reinforcement at



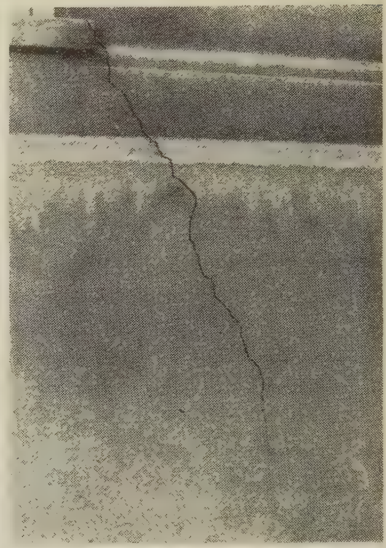


FIG. 5—CONTRACTION CRACK AT WINDOW CORNER DUE TO REDUCED SECTION AND TO CONCENTRATION OF SHRINKAGE STRESSES

FIG. 7—(RIGHT) CRACK AT WINDOW CORNER CAUSED BY UNYIELDING CHARACTER OF FORM AT SIDE OF WINDOW OPENING

the restricted sections. It is possible that the large cracks shown in both Fig. 5 and 6 could have been prevented by this method.

Although the use of contraction joints is the most positive method of relieving shrinkage stresses in concrete structures, their use may not be desirable in some cases because of their appearance, difficulty of construction, or for other reasons. However, it is possible to make a contraction joint fit into the architectural treatment of the structure in such a way that the presence of the joint will not be noticeable. It is even possible to omit the contraction joint but to provide definite sections of weakness at the base of a groove which is part of the architectural treatment, or by some other similar scheme, so that cracking will occur at a definite section where it will not be conspicuous. The cracking of concrete side-walks is arranged in this way by marking grooves at regular intervals and allowing the walk to crack at these grooves, with the result that the cracks are rarely seen. The cracking of buildings, retaining walls, and other concrete structures could be provided for and concealed with equal success.

In comparison with the crack shown in Fig. 5, that in Fig. 7 is of a distinctly different type, from a different condition. The former, due

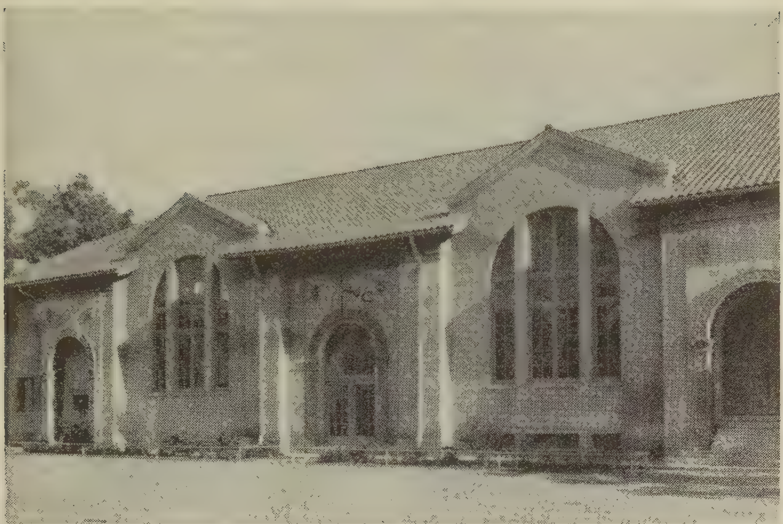


Fig. 6—Contraction crack at reduced section

to general contraction of the structure upon losing its moisture, was of approximately equal width throughout its length. In contrast the crack shown in Fig. 7 is a maximum at the corner of the window and disappears at some distance therefrom. Cracks of this type are partially due to the fact that the concrete in the wall or the wall column adjacent to the window cannot settle in the forms during the first few days as readily as the concrete of lesser restraint beneath the window. A differential shrinkage therefore exists between the concrete below the window and that beside the window, and this produces a maximum tensile stress in the concrete along a plane sloping downward from the corner of the window at an angle of about 25 deg. from the vertical, the stress being a maximum close to the window. Within a month or so after stripping the forms, the additional contraction due to loss of moisture raises the stress above the tensile strength of the concrete, so that a crack results. However, the crack extends only for a short distance as very little stress exists a few feet from the opening.

Cracks of this type resulting from the unyielding character of the form, which does not permit the concrete to adapt itself to shrinkage during the early period of hardening, have been successfully avoided by the use of a form at the side of the opening that can be released slightly about 12 to 15 hours after the concrete has been placed up to the top of the opening. This releasing of the forms removes the re-

straint of the concrete beside the opening and permits it to settle freely without being subjected to shrinkage stresses.

Contraction cracks of the types shown in Fig. 5, 6 and 7 can be almost entirely prevented provided flexible columns be used in the first story of the structure to relieve the shrinkage stresses. An inspection of structures having flexible lower columns will show an almost complete absence of shrinkage cracking in comparison with structures having less flexible lower stories. Laboratory tests on small models of the facades of various types of structures have clearly shown this beneficial effect of flexibility of the lower columns.

Buildings having a reinforced concrete frame and faced with masonry of stone or terra cotta, oftentimes have their appearance and even their usefulness greatly impaired because the masonry facing tied to the frame is subjected to excessive stress. This stress is usually developed by combinations of the three following factors: First, the contraction of the concrete frame with time subjects the masonry to compressive stresses which oftentimes result in severe cracking of the facing, thus making it unsightly and even permitting the entrance of water into the structure; second, the difference in thermal coefficients of expansion of the frame and of the facing, and even the difference in temperature of the frame and facing, due to the frame being subjected to a more or less constant moderate temperature whereas the masonry facing may be subjected to very low temperatures in winter and to high temperatures in summer, result in volume changes which are not the same for these two parts of the structure and thus produce stresses in both the frame and the facing; and third, tests have definitely shown that masonry laid up with cement mortars continue to expand for several months even though in dry air at constant temperature, and when subjected to moisture from rains the expansion is even more marked. This expansion, which is usually counter to the changes occurring in the frame, sets up severe stresses which are likely to damage the building.

To avoid this condition, the first item to be considered is the reduction of the volume changes in the concrete frame by proper selection of the concrete materials and the design of the mix. The volume changes in the facing may be likewise decreased by the use of leaner mortars than those oftentimes employed and by attention to the type of lime included in the mortar, as high magnesium limes produce greater volume changes, at least in brick masonry, than do the high calcium limes.

As it is impossible to prevent all volume changes in the frame and facing, it will also be necessary to introduce some type of pressure-

relieving joint at frequent intervals, preferably one in each story, which will permit displacement of the facing with respect to the frame and yet not permit moisture to gain access to the masonry backing.

A rather unusual case has been observed of cracking of the reinforced concrete frame of a one story manufacturing plant in Oakland, California, caused by the contraction of the frame and the simultaneous expansion of the clay tile curtain wall. The tile wall was laid up solidly from the floor to the underside of the roof beams. In a short time, the concrete columns had contracted and the tile wall had expanded sufficiently so that the reinforced concrete columns were subjected to high tensile stress and several of them developed very noticeable cracks, some of which were so wide that a knife blade could be inserted as far as the reinforcement steel. Not only were the columns subjected to stresses which were counter to those which normally obtain, but the roof beams were likewise affected. Instead of carrying loads which would tend to deflect the beams downward, they were subjected to an upward thrust by the expansion of the clay tile curtain wall so that cracks appeared in the beams on the top side toward the center of the spans.

A special method for reducing volume changes, which may be applicable in certan instances, is one involving the use of absorptive forms. Tests have demonstrated that the use of forms which withdraw the excess moisture quickly from the mortar or concrete before the initial set has occurred, tends to reduce materially the contraction of the concrete which develops after the mass has acquired a rigid form. The reduction in shrinkage for some mixes may even be greater than 25 per cent.

The Fountain of Time, designed and sculptured by Lorado Taft and constructed by John J. Earley, is an example of what may be accomplished by this method. This remarkable statuary is of concrete, with a length of about 120 ft., and although it is of intricate form and subjected to the extremes of temperature which occur in Chicago, it has not developed any cracking from volume changes. This is undoubtedly due partially to the fact that in constructing the statuary, a very lean porous mortar over the metal lath structure which forms the hollow core of the statuary was used to absorb the excess mixing water from the concrete. The use of aggregate consisting largely of quartz was a second factor which reduced the contraction in the mass, as by reference to Table 1 it will be seen that the contraction in air of quartz concrete is only about half that for crushed sandstone, concrete or for gravel concrete containing many sandstone particles.

In building construction it may be possible to introduce the use of absorbent forms instead of the usual wood forms, to remove the excess water from the mix and thereby to reduce the later volume changes. Another advantage of the use of such forms would be the ideal curing conditions which the absorbent forms would provide. In general, it will be appreciated that the use of absorbent forms probably has a limited application because of the added cost involved and because in some cases the rapid withdrawal of moisture from the mass will cause it to stiffen so quickly that difficulty may be experienced in proper placing.

Objectionable cracking in floors of buildings is often caused by too wet a mix. For satisfactory service, concrete floors should be put down as dry as adequate placing and finishing will permit. The use of enough water to cause the concrete to flow into place without working is to be condemned, as subsequent cracking and dusting is almost certain.

Massive concrete structures, such as dams and bridge piers, present a somewhat different problem than do those of thin section, because of the high temperature developed during the process of hydration of the cement. In very massive structures, the later temperature drop from the maximum setting temperature is of even greater importance in producing shrinkage than is the loss of moisture, as the latter proceeds very slowly. Setting temperatures have been observed as high as 150°F. in dams made of concrete containing 4.5 sacks of cement per cubic yard. As the concrete cools from this high temperature, large contraction occurs which causes excessive tensile stresses in the concrete and, in general, produces several contraction cracks throughout the mass. The later shrinkage due to loss of moisture causes additional contraction so that the cracks become wider and new cracks appear as the mass dries out. These cracks have long been a source of worry for engineers concerned with such structures. In the older structures, no provision was made, as a rule, to care for the shrinkage which was sure to result.

In constructing the Cross River dam in New York during 1906, an attempt was made to solve the problem by placing several 1¼-in. square steel bars in six layers toward the top of the dam. It was not expected that these rods would entirely eliminate the contraction cracks, but it was hoped that they would prevent concentration in a few large cracks. However, it was found that this reinforcement, both as to quantity and method of placing, was not effective in preventing the concentration of the cracks.

In more recent dams, contraction joints have usually been provided to control the location and character of the cracks and to provide against leakage. In some cases, these joints have not been spaced closely enough—for example, in the Lake Spaulding dam, the contraction joints were placed 80 ft. apart and yet shrinkage cracks developed during the first winter after completion of the dam, midway between each two joints.

The failure of the St. Francis dam, which was constructed without contraction joints, afforded an excellent opportunity to observe not only how a large concrete dam develops transverse cracks (which in this case were from 50 to 75 ft. apart in approximately radial planes) but also furnished evidence of the formation of inclined longitudinal cracks in the interior of the dam. These inclined shrinkage cracks were noticeable in the central portion of the St. Francis dam which remained standing. (See *Proceedings*, American Concrete Institute, Vol. 26, p. 83.)

It is now becoming common practice in the construction of high dams not only to provide transverse contraction joints but, to forestall and prevent the opening of cracks, the dams are built in sections or blocks, separated by joints normal to the faces of the structure. Alternate sections are built to considerable height, while those lying between are left a considerable time for later construction. After the sections first constructed have completely hardened and lost their chemical heat, and so far as practicable in the latter part of the winter, while the cold of winter has chilled the entire body of the sections built, and reduced them to a minimum volume, then the intervening alternate sections are built. The latter should be of materially smaller volume than the former, so that the chemical heat developed by the setting of the cement and the later contraction caused by loss of this heat will be correspondingly small. By this procedure, the whole structure is placed in compression during the heat of summer and the results show that the joints do not materially open when winter contracts the volume of the masonry. In addition to preventing leakage, the condition of high compression thus induced increases the tendency in the case of a straight dam to act as a beam, and in the case of a curved dam to act as an arch.

A procedure to care for the transverse shrinkage by the use of inclined longitudinal contraction joints has been suggested by Noetzli. This is described in detail in the *Proceedings* of the American Concrete Institute, Vol. 26, p. 84.

Still greater precautions seemed necessary to provide for the volume changes of such large masses as in Boulder dam. Due to its massive-

ness (725 ft. high and 650 ft. wide at the base), evaporation of the free water will proceed at a very slow rate so that shrinkage due to loss of moisture will probably not be a very serious problem, but one very important factor would be the very appreciable contraction of the mass while cooling from the high setting temperature certain to result. To cool the mass quickly and thus bring the structure to a condition of equilibrium so that the resulting contraction cracks may be grouted as soon as possible, special refrigeration methods involving the circulation of cooling water through a system of pipes imbedded in the concrete are being used.

For such discussion of this paper as may develop, readers are referred to the JOURNAL *for September, 1934 (Proceedings, Vol. 31). Discussions should be available to the Secretary by June 1, 1934.*

Ultimate Strength and Modulus of Elasticity of High Strength Portland Cement Concrete

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From time to time in the last few years references have appeared in engineering literature to concrete of a strength much higher than that in ordinary present day use. Speculations have been indulged and computations made as to the advantages from the use of such concrete. The Albert Louppe bridge in France is reported[1] to have used a concrete with an ultimate strength of 8500 p.s.i. (probably for 8 in. cubes, 90 days old). Published data on the properties of concrete of an ultimate strength greater than 6000 p.s.i. are meager. It is hoped, therefore, that the following material will prove of interest.

To investigate the upper limits of strength of portland cement concrete and Young's modulus in compression for these higher strengths, a series of tests was inaugurated about a year ago at the University of Colorado. In these tests, still in progress, these variables are being studied:

1. coarse aggregate,
2. c/w ratio by weight,
3. age at test.

The mortar used in the specimens was made of Ideal portland cement manufactured at Boettcher, Colorado, and a good local washed sand, 0-4 grading. Coarse aggregates were all local materials: a gravel with specific gravity = 2.63 and percentage absorption = 0.6, a basalt porphyry with specific gravity = 2.75 and zero percentage absorption and two sandstones of the Lyons formation, one red and the other white. Although both are called Lyons sandstones they have somewhat different physical properties. The red sandstone has a specific gravity of 2.50 and a percentage absorption of 1.5; the white has corresponding values of 2.56 and 1.0. All coarse aggregates were

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†Respectively, Asst. Prof. and Assoc. Prof. of Civil Engineering, University of Colorado.
[1]Eng. News-Record, Oct. 1929.

graded in the following proportions: 25 per cent, ¼-⅜, 25 per cent ⅜-½, 50 per cent, ½-¾.

The method of proportioning advocated by Inge Lyse[2] and others was followed: This is based upon the principle that the ultimate strength varies linearly with the "Concentration of cement particles" per unit of concrete. (This means that if ultimate strengths are plotted against c/w ratios by weight rather than w/c ratios by volume the graph is a straight line instead of the familiar Abrams curve.) Furthermore it is claimed that mixes of various ultimate strengths when proportioned according to certain conditions have the same slump. These conditions are that the mixing water be kept constant per unit of volume of concrete, that the gradation of aggregates be kept constant, and that the absolute volume of combined materials be kept constant for all mixes. In addition to the main objectives of these experiments already stated the authors were interested in testing this method of proportioning.

A ratio of fine to coarse aggregate of 1:1½ by weight was determined from a trial mix using the red sandstone as coarse aggregate. The proportion varied slightly for the other coarse aggregates due to the difference in specific gravity. Under the method of proportioning outlined above the lower strength mixes proved to be under-sanded.

For concretes made with the red and white sandstones the c/w ratios were 2.0, 2.5, 3.0 and 3.5. Specimens using gravel and basalt were all of a single c/w ratio of 3.0. In an effort to get a very rich concrete a mix with c/w ratio equal to 4.0 was tried but found too dry for mixing by ordinary methods. Water used was at the rate of 38

TABLE 1

Series	Coarse Agg.	W/C Ratio (by Vol.)	C/W Ratio (by Wt.)	Bags per cu. yd.	Slump	Average Compressive Strength	
						28 Days p.s.i.	90 Days p.s.i.
RD	red sand-stone	.75	2.0	6.7	1.0	4550	5570
RE	red sand-stone	.60	2.5	8.4	0.6	6500	7710
RF	red sand-stone	.50	3.0	10.1	0.6	8710	9990
RG	red sand-stone	.43	3.5	11.8	0.2	9720	10470
WD	white sand-stone	.75	2.0	6.7	1.0	3940	5530
WE	white sand-stone	.60	2.5	8.4	0.8	6070	7630
WF	white sand-stone	.50	3.0	10.1	0.8	8350	9810
WG	white sand-stone	.43	3.5	11.8	0.2	9560	11080
GF	gravel	.50	3.0	10.1	3.0	8000	9070
BF	basalt	.50	3.0	10.1	0.8	10150	12000

gal. per cu. yd. Test specimens were 3 in. by 6 in. cylinders. Results are shown in Table 1 and Fig. 1. Values given are the average of at least five specimens.

[2]Eng. News-Record, Nov. 5, 1931 and Feb. 18, 1932.

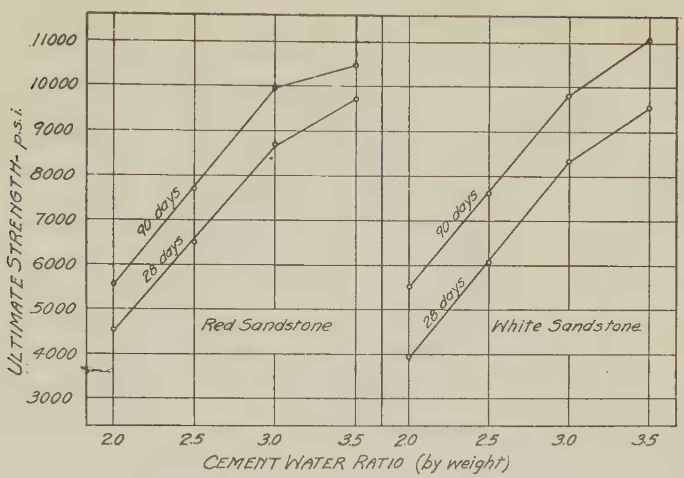

FIG. 1—RELATION BETWEEN ULTIMATE STRENGTH AND CEMENT WATER RATIO

Curing was continuous under water and specimens were tested at ages of 7, 28, and 90 days. The cylinders were capped just before testing with a mixture of three parts sulphur and one part fire clay heated to form a liquid. This method of capping was found to be satisfactory. The caps withstood pressures of 13000 p.s.i. without any visible signs of distress. Some of the cylinders of each series were tested in a 60,000 lb. hydraulic testing machine at a speed of 7 p.s.i. per second, and the rest in a 150,000 lb. screw-type machine at a speed of 70 p.s.i. per second. Careful study of the results obtained from specimens of the same mix failed to indicate any variation due to the different rates of loading.

From Fig. 1 it can be seen that the ultimate strength at 28 days and at 90 days may for all practical purposes be said to vary linearly in proportion to the "concentration of cement particles" up to a value of c/w = 3.0 or slightly greater. The additional strength obtained by bringing the c/w ratio to 3.5 is not proportional to the increase at lower ratios. Other experiments by the authors show that the straight line relationship holds between values of c/w = 1.0 and c/w = 3.0.

Table 1 shows the slump to be approximately the same for the mixes with crushed rock aggregate and greater with the mix using gravel. This agrees with results found by other investigators. It should be noted that there is a tendency for the slump to increase in the lean mixes and to decrease in the rich. This tendency has been noted also

in other experiments of the authors. It has been found that when the mixes are proportioned in the manner described, those with the smaller amounts of cement tend to be harsh and to have a slightly greater slump. The extremely high-strength mixes, on the other hand, are inherently stiff and have a lower slump.

If the RG and WG series ($c/w = 3.5$) are ignored as being uneconomical in the use of cement it is found that the highest ultimate strengths, obtained at 90 days were for concretes with

gravel	9070 p.s.i.
white sandstone	9810 p.s.i.
red sandstone	9990 p.s.i.
basalt	12000 p.s.i.

Although these tests are in themselves far from conclusive it is reasonable to suspect from the results obtained that for very high strength concretes the ultimate strength attained depends to some extent upon the kind of coarse aggregate used. This may be due wholly or in part to one or both of two factors: the strength of the aggregate against shearing forces and its ability to develop a high degree of bond with the mortar due to the texture of the surface. Most of the specimens broke with the usual cone-type of fracture although some had one main shearing plane. In both cases, the planes of fracture were fairly clean-cut passing through whatever pieces of rock or gravel lay in the path of cleavage. It would seem likely, therefore, that the strength of a specimen is dependent at least to some degree upon the resistance which the pieces of coarse aggregate crossing the planes of fracture can offer the forces causing failure. As a matter of fact in these experiments the specimens, if classified as to the soundness and apparent strength of the aggregate fall into the same grouping as when classified according to ultimate strength. Under the blows of a hammer the gravel proved to be the weakest aggregate, the basalt the strongest and the sandstones intermediate between the other two. The basalt was a particularly dense stone which absorbed no water during an immersion of four hours. It was broken only by very sharp blows of the hammer.

It is possible too, that the fracture of a cylinder at ultimate load may begin by a local failure of bond between mortar and coarse aggregate, the failure then progressing along the plane of fracture. From this point of view it is to be expected that the relatively smooth surface of the gravel would afford less bond than the rougher surfaces of the other aggregates. One would not, however, expect the basalt to offer sufficiently greater bond than the sandstone to account for the large increase in strength to 12,000 p.s.i. Further research to study

the part played by the coarse aggregate in determining the ultimate strength of the concrete should prove highly interesting.

Axial stress-strain measurements were taken to determine the modulus of elasticity. The compressometer was of the 2-yoke 5-clamp type used by the U. S. Bureau of Reclamation in its Denver, Colorado laboratory. The same type of instrument has been used by Prof. H. J. Gilkey[3], Stanton Walker[4] and others. A "Last Word" dial reading to 0.0001 in. was used with the compressometer.

Many investigators have reported the stress-strain diagram as being a curve of decreasing slope, one which could be expressed in the form of the equation, $S = Kd^n$, where S = unit strength, d = unit deformation, K = a constant depending on the ultimate strength of the concrete and n = an exponent approximately constant.[4] The concrete upon which such results were based have been of ultimate strengths varying ordinarily from 1500 p.s.i. to 4000 p.s.i. with occasional values somewhat higher. It has been found that the curvature decreases as the ultimate strength increases. Similar results were obtained in these experiments for ultimate strengths between 2000 and approximately 4000 p.s.i. For higher strengths, however, the curve becomes definitely a straight line up to the proportional elastic limit which occurred at approximately 50 per cent of the ultimate strength. It may be said, then, that when the entire family of curves is studied as a group with ultimate strengths varying from 2000 to 11000 p.s.i. it is found that the curves in the lower strengths have a fairly large curvature, that they tend to straighten out as the ultimate strength increases and finally emerge in the higher strengths as straight lines up to the proportional elastic limit.

Fig. 2 shows the relation of the modulus of elasticity to ultimate strength. Where the stress-strain curve was not a straight line the modulus was taken as the slope of the tangent to the curve at 25 per cent of the ultimate strength. Each plotted point represents the value determined from an individual specimen; no average values are plotted. First tests and immediate retests are included. (On the red sandstone series only a few retests were made.)

Fig. 2 is interesting in two respects. A definite relationship is shown between ultimate strength and modulus of elasticity throughout the whole range of concrete strengths, from an ordinary mix of 2000 p.s.i. to a strength of 11000 p.s.i. In addition the points group themselves into two distinct paths, approximately parallel, one for the concrete with red sandstone as coarse aggregate, the other for the

[3]Proc. A. S. T. M., Vol. 30, Part I, 1930.
[4]Proc. A. S. T. M., Vol. 9, Part II, 1919

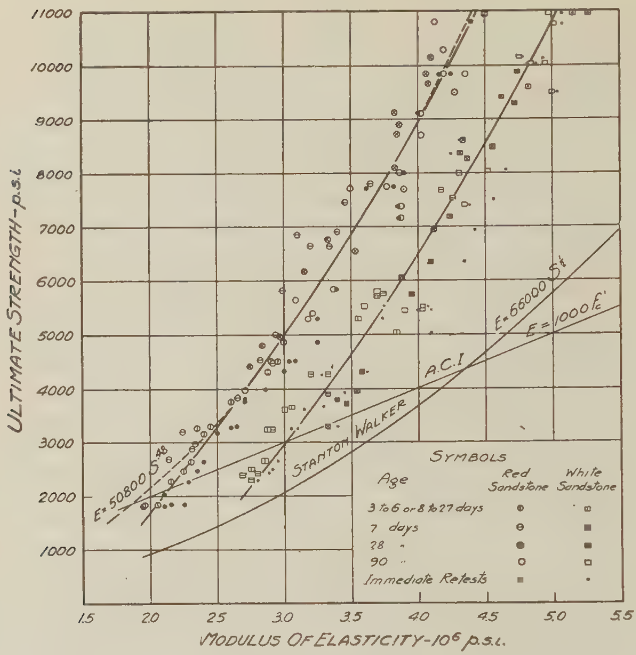

FIG. 2—RELATION BETWEEN MODULUS OF ELASTICITY AND ULTIMATE STRENGTH

concrete with white sandstone. As previously mentioned the two sandstones have different physical properties although both are of the Lyons formation. That the modulus should vary with the coarse aggregate seems reasonable. J. W. Johnson using two kinds of aggregate, limestone and gravel, found this to be true,[5] although Stanton Walker[4] did not. It would appear, however, that the coarse aggregate forms too great a proportion of the finished material not to affect the final physical properties. If, for example, it were composed of pieces of hard rubber in one instance, or of steel in another, it is natural to expect the modulus of elasticity of the two kinds of concrete to be appreciably different. In these experiments, no attempt has been made as yet to determine how close the relationship is between the modulus of elasticity of the stone used as coarse aggregate and that of the finished concrete.

In Fig. 2 the curve for the concrete with red sandstone was drawn and then that for the white made parallel to it; that is to say E on the latter curve as drawn is greater than E on the former for all ultimate strengths by the value of 600,000 in.[4]

[5]Bull. 90, Eng. Ex. Sta. Iowa State College.

The equation for the curve for red sandstone is

$$E = \left[1.43 + 0.345 \frac{S}{1000} - 0.0065 \left(\frac{S}{1000}\right)^2\right]10^6$$

and for white sandstone is

$$E = \left[2.03 + 0.345 \frac{S}{1000} - 0.0065 \left(\frac{S}{1000}\right)^2\right]10^6$$

where E is Young's modulus in inches[4] and S = ultimate strength in p.s.i.

These equations are equations of parabolas. It is interesting to compare them with the equation suggested by Stanton Walker as applying to all concretes regardless of the kind of stone used for coarse aggregate, viz: $E = 66000S^{1/2}$. This curve is shown in Fig 2 and it falls entirely out of the range of results determined from these tests. A curve, of the same exponential form but with different constants, $E = 50800\ S^{.48}$, fits the results for the concrete with red sandstone very closely except for the lower strength concretes; a similar curve could easily be drawn for the white sandstone. The A. C. I. straight line formula, $E = 1000\ f_{c'}$ is also shown on Fig. 2. It is much flatter than the parabolic curves and gives values of E much greater than seem warranted for the higher strength concretes.

In studying the relation between the ultimate strength and Young's modulus as determined in these experiments, one should remember the conditions under which the mixes were determined; namely, that for all mixes the gradation of aggregates, the absolute volume of the constituent materials and the quantity of water per unit of volume of concrete were all kept constant. It is possible that under other conditions of proportioning different results would have been obtained.

CONCLUSIONS

1. Ultimate strengths as high as 12000 p.s.i. were obtained for 3 in. by 6 in. cylinders cured 90 days under water and tested wet. Specimens of the same mix at 28 days averaged 10150 p.s.i.

2. Mixes yielding 8000 to 10000 p.s.i. (at 28 days) were fairly stiff but there was no difficulty experienced in casting sound cylinders in the laboratory with the assistance of hand tamping.

3. Mixes were designed on the basis of a constant quantity of water per cu. ft. of concrete, a constant grading of fine to coarse aggregate and a constant absolute volume of constituent materials. Under these conditions the ultimate strength may be expressed as a straight line function of the c/w ratio (by weight) up to the region where c/w = 3.0. Above this point the strength increased at a lower rate with

increased c/w ratio. A mix designed for $c/w = 4.0$ was too stiff to handle by ordinary methods.

4. For very high strength portland cement concrete it appears that the ultimate strength is dependent to some degree upon the strength of the coarse aggregate, a very dense rock seeming to assist in raising the strength against final failure. It is possible, also, that the ability of the rock to bond itself to the mortar affects the ultimate strength. Much further research is needed on this problem.

5. For low strength concrete the stress-strain curve is the familiar curve of decreasing slope. As the ultimate strength is raised the curve becomes steeper and at the same time approaches a straight line. In the neighborhood of 4000-5000 p.s.i. the curvature is scarcely discernible and for higher values the curves is definitely a straight line up to the proportional elastic limit. The latter occurs at about 50 per cent of the ultimate strength.

6. The modulus of elasticity varies with the coarse aggregate used. Under the conditions of these tests the change from one sandstone to another sandstone of different physical properties increased the modulus approximately a constant value of 600,000 p.s.i. throughout the whole range of strengths from 2,000 to 11,000 p.s.i.

For such discussion of this paper as may develop, readers are referred to the JOURNAL *for September, 1934 (Proceedings, Vol. 31). Discussions should be available to the Secretary by June 1, 1934.*

Structural Design of Baha'i Temple*

BY BENJAMIN B. SHAPIRO†

My first contact with the design of the Baha'i Temple was in association with the late Major H. J. Burt, in 1921, when I was delegated to work with the late Louis Bourgeois to prepare the structural design. It was divided into two divisions—foundation and super-structure. The super-structure was to be that portion of the building above the first floor, the design to be preliminary in form, enough to determine structural reactions and loads. The sub-structure design was to be final, for contract. The whole design was to be complete enough so that a building permit could be taken out for the structure as a whole. As a general description of the Temple has been presented to you** the structural design as presented will be similar to a log of the construction of the building in its various components.

FOUNDATION

The foundation problem is a somewhat intricate one. There are heavy loads at nine points which support the main dome. At the other points the loads are comparatively light, carrying as they do only one floor and the roof, together with the walls. As a matter of sentiment as well as of safety, it was desired to support the dome on bed rock. On this basis the foundations of the dome consist of nine piers extending to rock at a depth of 124 ft. below the ground level. For the comparatively light loads outside of the dome area so far as the mere support of the weight is concerned, it would have been permissible to use spread footings, but for greater security against settlement, and particularly in view of the fact that the main central portion of the building would be on rigid foundations extending to rock, it was decided to use concrete piles for a support of the structure lying outside of the central portion.

While it is not the best practice to mix two different types of founda-

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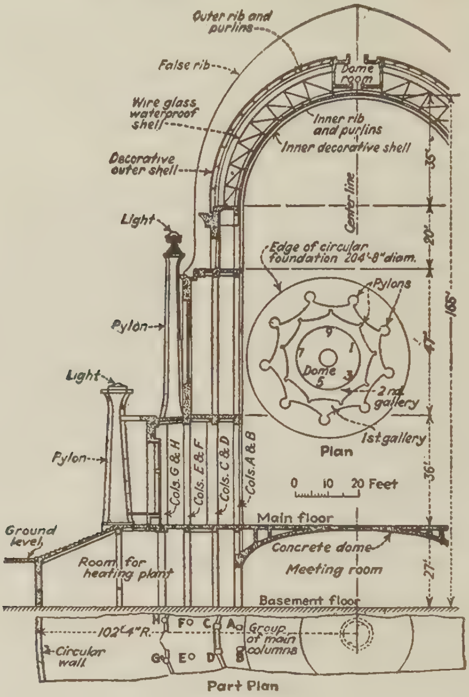
**"The Temple of Light," by Allen B. McDaniel and "The Project of Ornamenting the Baha'i Temple Dome," by John J. Earley, Journal, Amer. Concrete Inst., June 1933, *Proceedings*, Vol. 29, p. 397 and 403.

tions in a structure, it is believed that both of them are secure enough so that no movement can occur to mar the super-structure.



TYPICAL SECTION OF TEMPLE AND DOME

Courtesy *Engineering News Record*, Jan. 8, 1931—in a description of the structural design.

In both types of foundations difficulties were encountered. Water was the great difficulty with the foundations extending to rock. These were constructed in wells 6 ft. in diameter. At a depth of about 80 ft. below the surface, water was encountered in considerable quantities so that the work could proceed only very slowly. At times three steam pumps were used in a single well to remove the water fast enough to permit digging. All of the wells were finally landed successfully on bed rock and filled with concrete.

At the top of the pier the concrete is flared to a rectangular shape 9 ft. 9 in. x 11 ft., to carry four continuous columns which support the dome, galleries, and portions of the first floor framing. These columns rest upon combined I-beam grillages in two courses, which transmit a load of approximately 700 tons to each pier. The main shaft of the pier was specified as 1:2:4 mix; the flared top a 1:1:2 mix.

For the pile foundations, patented concrete piles were used, and were supposed to be in accordance with the contractor's specifications, to give certain definite results. After these piles had been driven, some of them were tested, and it was discovered that they would not carry safely the designed loads. In each case the test load was twice the computed ultimate load. After several tests, it was evident something was seriously wrong, and it developed that the piles were not put in in accordance with the contractor's claims and it became necessary to reinforce all of the minor footings. This was done by driving additional piles and extending concrete caps to embrace them. These were composite piles: wooden piles 35 ft. long were driven to such depth that the top of the wooden pile was approximately 20 ft. below the ground line. Then a concrete shaft 16 in. in diameter, was placed on top of the wooden pile, extending to the under side of the concrete footing. These new piles were tested and found to have ample supporting power.

SUB-STRUCTURE

The sub-structure is 204 ft. 8 in. in diameter and 27 ft. high. Architectural requirements were such that the tasks of the structural designer were increased. The outer 26 ft. of the sub-structure was sloped to take care of the ultimate circular stairs, the outer wall being 15 ft. 9 in. high, and the stair rise 11 ft. 3 in. Fill will bring the ground level to this height.

To have a central basement space 72 ft. in diameter, the ceiling and floor above had to be supported without interior columns. To provide this support it was decided to use an elliptical reinforced concrete dome. Design of the concrete dome was developed by means of graphics and checked analytically. Its shell is 12 in. thick; reinforced with two layers of steel rods of ¾ in. diameter, 12 in. on center, one near the top and one near the bottom. Each of these layers is made up of rods in radial position, and others of ¾ in. and of ⅞ in. diameter, 12 in. on center, circumferentially. This dome in turn rests upon a circular concrete girder 21 in. wide and 48 in. deep, carried in turn by the encasement of the steel columns which rest upon the concrete piers to rock.

Upon the shell of the dome 6 in. stud walls of reinforced concrete were built to carry the floor slab having maximum clear spans of 5 ft. 6 in. An opening of 10 ft. diameter was left in the center of the dome for prismatic lights. The sub-structure is of reinforced concrete except for the nine sets of structural steel columns carried by the nine concrete piers.

In placing the concrete for the dome, a very dry mix was used which had to be rammed. There was no excess water, and after placing no tendency to flow was observed. Extreme care was taken to get a proper grading and mix, and no excess water was permitted. And while water-cement ratio did not then govern practice, the precautions and requirements of water-cement ratio were observed unknowingly.

This sub-structure was subjected to the elements for nine years, the top surface of the structure not finished in any way, and at no time has there been found any deterioration due to weathering.

SUPER-STRUCTURE

In 1930 it was decided to proceed with the super-structure. A careful analysis was made of the architectural plans, and it seemed that the Architect had decided that straight lines were obsolete except for vertical columns. Everything was curved, and a composite structure of structural steel and reinforced concrete was found to accommodate all of his designs.

At the first floor level the building extends beyond the barrel of the dome, the area of this portion being several times the area of the dome itself. This provides the principal usable space in the Temple.

The curved walls and arches offered some problems in structural and form work design.

The main design feature consists of a clear circular opening 72 ft. in diameter and 105 ft. high above the main floor, surmounted by a dome of 36 ft. radius. There are no intermediate floors, but at various levels there are galleries circumscribing the circular open space. The base at the first floor is 36 ft. high and approximately 150 ft. in diameter, divided into nine alcove units with a main building entrance between each of the units. This base in turn is surmounted by the first gallery level 47 ft. high and of approximately 136 ft. outside diameter. Above this is the second gallery level 20 ft. high and of approximately 93 ft. outside diameter. One hundred three feet above the first floor a third gallery, actually the base of the dome, forms a level for service and inspection of utilities placed thereon.

Each level, divided into nine units, will be emphasized by ornamental pylons at each of the intersections of the sectors. Those at the first story, 47 ft. high, those at the second gallery 52 ft. high. The pylon effect is continued from the second gallery in false ribs, extending over and becoming a part of the ornamentation of the exterior dome, and meeting at a point 162 ft. above the first floor level.

The complete exterior ornamentation of the structure will be of concrete tracery, backed by glass. This tracery will extend from the base at the first floor to the top of the dome. Under this exterior dome tracery is placed a waterproof shell of wire glass, and below this an ornamental perforated interior shell forming the dome of the inner central feature.

The lighting features which are necessary for novel effects, will be between the glass water shed and the interior tracery.

In the steel polygonal superstructure, each of the nine corners or the intersection of the sectors consists of a group of four 14-in. H-columns with struts and diagonal bracing which are continuous for the support of the galleries and the dome and four additional H-columns and additional concrete columns for the support of the roofs over the first floor.

At the first and second gallery levels, to form the circular well opening in the center of the building, circular steel channel girders are introduced framing on inner and outer face of the interior columns which are on a radius of 37 ft. 9 in. from the center of the building. These circular girders are designed on the basis of full continuity and are further reinforced by the means of plate and angle separators riveted between the circular members. The balance of the gallery floor framing is composed of standard steel framing all tied together by a reinforced concrete slab with a minimum thickness of 6 in. In the design of the first and second gallery floor framing, special framing has been provided for the support of the pylons and the false ribs that continue over the dome.

In the first story and first gallery the walls are curved inward and arched over the window openings.

Cantilever girders of steel were introduced in the first and second gallery floor framing to carry the greater portion of the load, and thus relieve the piers on each side of the pylons.

For the first story arches, the cantilever girders are placed on approximately 14 ft. centers at the center of the arch, and carry 50 per cent of the arch load.

For the first gallery arches the cantilevers are at the end of the arches and carry approximately 75 per cent of the load.

At the third gallery level, two circular girders have been provided for the support of the exterior and interior dome framing. These

circular plate girders are framed to the faces of the columns that are continuous to the cassions and are securely tied together by means of steel framing composed of I-beams and gusset plates, and a minimum concrete slab of 5 in. thickness so as adequately to take care of any torsional stresses. The exterior circular plate girder is 36 in deep, with a radius of 44 ft. 8½ in. The inner girder is 24 in. deep with a radius of 38 ft. 4¼ in.

The walls and pylons are all of reinforced concrete, all shaped and formed to meet with the ultimate architectural design so as to give an approximate idea of the final design of the building.

In design very careful consideration was given to the matters of expansion and contraction, lest cracks should damage the future ornamentation. It was found that the architectural requirements were such that they fitted most ideally into the scheme. Each of the pylons was designed as a hollow circular expansion ring instead of a flying buttress, and acted as an expansion shell between the main section of the walls. In this way no cracks were formed during the construction, and none has been observed on the structure to date.

While ultimately the concrete of the super-structure is to be given an ornamental covering, it was not known when this could be done. Exposed to the severe weathering action which occurs on the shores of Lake Michigan, durable concrete was an important factor.

Great care had to be exercised in proportioning the mix of the concrete because the workability had to be varied in relation to the size and construction of the different members. Placing of concrete for window mullions and columns was a task, for it was impossible to work the concrete in these members with ordinary spades or rods. The columns and mullions are from 30 ft. to 47 ft. high, and in some instances the width only 4 in. Rubber mallets were used to vibrate the forms to get maximum density and to cover the reinforcing bars.

Increased workability of concrete was obtained for structural members of thin section by reducing the proportion of coarse aggregate in the mixture.

The entire responsibility for obtaining the desired strength of concrete and the varying of the workability for mixes, so as to conform to the various conditions as well as the proper curing of the concrete, was in the hands of the engineers and representatives of a testing laboratory. Representatives of the Portland Cement Association co-operated in every way so that the final results obtained would be adequate.

While the plans and specifications call for a concrete by water-cement ratio, to be of a minimum strength of 2000 p.s.i., after selecting and analyzing the aggregates, a mix was designed for a minimum strength of 2500 p.s.i., taking into consideration the cement and the amount of water. Screen analyses of the aggregates were run daily to insure uniformity. The fineness modulus of the sand varied from 2.7 to 3.15, and that of the stone from 7.3 to 7.5.

A slump of 7½ in. was used on thin sections, 5 in. on other portions of the work. The yield obtained from one sack of cement was 5.6 cu. ft. of concrete, so that the cement factor was 4.82 sacks to the cu. yd.

The average 28-day compression test result was 2900 p.s.i., and the average 7-day compression test result was 1500 lb. Accurate water control was maintained by using a barrel of uniform diameter in which was placed a graduated rod. Mixing time was 1½ minutes per batch.

Two test cylinders were made daily, one for 7 day cure on the job, and one for 28 days, cured in the laboratory. The 7-day test was of assistance in determining when it was safe to strip forms. Where possible forms were left in place for a longer period, especially on the thin sections, as a curing aid.

Since a large portion of the concrete was placed in mid-winter, protection had to be provided. Salamanders and canvas covering were used. Due to the height of the columns and the location of different members, it was difficult to protect freshly placed concrete in freezing weather, and it was necessary to discontinue concreting when weather reports indicated the probability of temperatures below 20° F. Temperature of the concrete when placed ranged from 70° to 100° F.

Aggregates and water were heated by steam boilers. Live steam was used in cleaning the forms and steel reinforcements of ice and snow previous to depositing concrete.

DOME DESIGN

The structural design of the dome consists of an inner and outer concentric framing of ribs made up in an I-section, having two angles for each chord and lacing bars for web members. The vertical radius on the bottom chord for the outer framing is 44 ft. 8½ in. and for the inner framing 38 ft. 4¼ inches. Inner and outer systems are independent and un-connected so that unequal expansion and contraction will in no way affect the structure. The outer ribs frame against a

16 in. ring girder around a central opening 16 ft. in diameter at the crown of the dome. The inner ribs frame against a similar ring girder composed of a pair of 10-in. channels, back to back. A concrete slab at each of the ring girders forms a solid floor at each level. Ladders hang from alternate outer ribs, terminating in a catwalk and supported on the inner framing is also a system of catwalks and ladders—both for inspection and maintenance. The future ornamentation of the interior will be supported by hangers from the inner framing. From the inner rib is a series of angle struts for the support of the glass water shed.

RIB AND RING AND DIAGONAL STRESSES

In determining the rib and ring stresses, the method outlined by E. Schmitt*, with minor modifications, was found to be the most practical. Obtaining the diagonal stresses was more intricate. Several methods were considered, and that given by Hutte was adopted. Snow and wind loads were taken care of and this had to be considered on both the systems of framing, as the concrete tracery for the exterior shell would not completely retard the action of the wind and snow on the inner dome.

Extensive power equipment for heating, lighting, and ventilation is in the basement. Although the heating is provided by oil fired boilers, there is no smoke stack in or adjacent to the building. An underground tunnel has been provided to a stack 5 ft. in diameter and 30 ft. high, 244 ft. from the center of the building, all of which is operated by an induced draft system.

*Transactions, Am. Soc. of C. E Vol. 52, 1904.

For such discussion of this paper as may develop, readers are referred to the JOURNAL *for September, 1934 (Proceedings, Vol. 31). Discussions should be available to the Secretary by June 1, 1934.*

President's Address*

BY S. C. HOLLISTER

It is fitting that a periodic account of stocks be taken concerning the state of the Institute. I choose to view the state of the Institute this evening at this 29th anniversary of its founding not in the traditional light of receipts and disbursements, assets, liabilities, and surplus if any; but rather in the broader view of the Institute's work in relation to the field which it serves.

From a strictly business point of view it is pleasing to report to you that the Institute is in good health due in part to the far-seeing policies of your former officers to provide for an eventual rainy day and in part to the successful efforts of the present officers in conserving that cushion of protection against insolvency and obliteration in these trying times. The Institute has ridden the storm and has proven herself a sturdy ship. To the whole crew, to each of the membership, goes a measure of credit for his loyalty in this achievement. But perhaps to no single individual goes as much credit for this achievement as should rightfully go to your Secretary-Treasurer, Harvey Whipple, and to him I pay sincere and humble tribute. And so the Institute can and will go on to the service that lies ahead.

But my special theme this evening lies in the answer to a challenging comment passed to me not long ago. Have we not at hand essential rules for the proportioning of concrete to obtain strength, durability, water-tightness? Have we not elaborate equipment, many specifications and codes by which to construct concrete work? What need for additional data and what data to add? In short, has not the field been served and has not the Institute shot its bolt?

One may grow so accustomed to the surrounding conditions that they are accepted as a sort of status not subject to review. An automobile is ordinary when it is a daily necessity; but today's automobile would have been an unquestioned marvel twenty, or even ten years ago. The achievement of today was the goal of yesterday. It cannot be the goal for tomorrow. Great as have been the achievements in

*Presented by the retiring president, Prof. S. C. Hollister, at the Institute's 30th Annual Dinner, Toronto, Feb. 21, 1934.

the field of concrete today, they are only the dreams of yesterday come true.

If the function of the Institute has been fulfilled, it is because the realization of dreams is accomplished and because the realm of dreams has been wrung dry. If there is a field for the Institute it is because there still exist dreams of accomplishment—developments not yet realized.

Is the realm of dreams for the Institute really barren? In answer, who is there who would say that in any single major phase of the field we have reached what ten or twenty years hence we will expect to see as established practice?

But let us be more specific. Judging by the development of the last twenty years what do we think the cement of twenty years hence will be like? What will be the currently available concrete strength, and what the construction methods? In this frame of mind I have dared to review even the most complacently accepted practices of today and to ask, What reasonably possible developments may be made in the next twenty or even ten years?

There are many indications as disclosed by reports to this Institute that a number of interesting developments are possible in the basic product, cement. This is quite aside from the question whether one cement or several cements will be available. Surely a majority of concrete structures may be built of a standard cement in any case. But will it approach in character the present standard portland, or high early strength, or low heat, or still something else? Admittedly this is not so much a question to be solved by the Institute as it is a basis to which concrete work is to be adjusted.

And what of concrete mixtures? Abrams has made mortar cylinders with the strength of steel. Moreover, they behaved in accordance with the water-cement ratio law. To obtain this phenomenal result he had to use pressure to cause the minute amount of water to be dispersed throughout the cement. What a challenge this has offered! If this dispersion could be accomplished in other ways, or if only a part of this added strength is made available by easily applied methods, a new era in concrete is born. Imagine, for example, the concrete with an available strength of 10,000 pounds per square inch. Smaller columns, thinner and lighter beams and slabs would at once result. Precast units, easy to handle, would be available. The present limiting heights of buildings, of spans of bridges, would be at least double. A new basis of design, new codes and specifications would be required.

Perhaps it may seem to some that these are idle dreams, but let us see whether there is any tangible basis for thinking there is a possibility of moving on from our presently current strength to those of Abrams' cylinders of the strength of steel.

Present methods in use by Earley in the construction of the Baha'i Temple permit them to place their units weighing three tons or more and to lift them out and turn them over in eighteen hours after filling the mold. And this with portland cement, not with excessively rich mixtures but with much of the excess water removed *after* placing but *before* the setting of the cement. It is interesting to speculate on the effect upon concrete design and construction methods such a development would produce in the ordinary concrete field.

Talbot and Richart have contributed valuable information on the physics of wet mortar. This field of enquiry is a fertile one and will likely yield further developments. Who may say, for example, whether it is possible to achieve mobility or workability with an agent other than water thus using only enough water for the hydration of the cement.

In the realm of mass concrete we have a great deal to learn. Not only must there be much improvement in the understanding of the mechanics of large concrete units, but we must know more of the thermal and shrinkage stresses, and their relation to the chemical and physical changes transpiring. And to cap these effects we must determine more accurately the aggravating or mollifying effect of plastic flow.

We have by no means exhausted the possibilities in the development of mixing and placing equipment. Even now harsher, stronger mixtures are producible and placeable where in late years they would not have been considered feasible. Extracting the water from the wet mass has been known to be beneficial—in fact, Earley has used this principle for many years; but we have not as yet brought about the practice in general construction.

As new methods of experimentation become available much remains to be developed in the mechanics of reinforced concrete members. A review of the beam theory, especially at a cracked section, and after considerable plastic flow, will yield many useful data. A study of bond strength and of anchorages will lead to a better design procedure. The work of Davis, McMillan and others on plastic flow will give new color to the mechanics of reinforced concrete design. The manner in which shrinkage stresses relate to our accepted design procedure has not been fully determined.

A great deal of work is necessary for a better understanding of the mechanical behavior of many forms of commonly used members. This is especially true of the inter-action of members of an assembly. For example, in arch bridge design there are at present proponents of both the articulated deck and the monolithic deck. Abroad there are many developments in bridge design and construction which have not as yet been used here, notably the thin flexible arch with rigid deck frame and arches with hinges.

Many developments in structural form are to be seen in Europe. These may in many cases prove of use on this side of the Atlantic. To the ingenuity of the adaptation of structural form in concrete there is no end.

Valuable investigations by McMillan, Young, Viens, Lindau, and others, upon the factors influencing durability of concrete structures have from time to time been reported to the Institute. This work will of necessity go on for many years to come before we may be fully satisfied that the subject has been adequately covered.

In review then, we see the many varied and intriguing avenues of development that present themselves both for the immediate and the long time programs of action. We have not touched yet upon the products field. Little mention has been made, except by implication of questions relating to aggregates to curing methods and the interesting likelihood is that no sooner will we have caught up with one branch of our varied subject than the basic substance, concrete, or its parent, cement, will have moved to new levels.

Surely then, the work of this Institute is not at all done. Each year sees new accomplishments, and with them new objectives appear. Thus the Institute is but the embodiment of a moving force of progress. It cannot be static. It can never overtake its goal.

Architectural Concrete of the Exposed Aggregate Type*

BY JOHN J. EARLEY†
MEMBER AMERICAN CONCRETE INSTITUTE

This paper is the continuation of a paper presented last year to the Institute. The previous paper described the problem presented by the ornamentation of the Baha'i Temple[1]. This paper describes some of the technique by which architectural concrete of the exposed aggregate type has been developed and some of the methods by which the ornamentation of the Temple has been done.

As I look back over the work of our studio with concrete I see from year to year a noticeable improvement in its appearance. The work is better both in design and execution. The improvement has been continued and rational and in general what should be expected from a studio such as ours. Nevertheless, I am impressed that the most important improvements affecting the nature of the material did not come gradually from year to year but quickly, when the material was made to take on an added quality to meet the requirements of some particular job of work. It took on character, which was necessary for that work, which suddenly developed in the highly concentrated attention paid to the problem, which remained with the material after the experience had passed and which befitted it for a new order of use.

I have in a more or less disconnected way recorded in the *Proceedings* of the American Concrete Institute some of the most important developments. For instance: We developed for the work at Meridian Hill Park control of the appearance of concrete of the exposed aggregate type by means of a two-step gradation of the aggregate.[2] Upon this theme rested all future development of this type of architectural concrete. We reasoned that if every particle of stone exposed upon the surface of the concrete might be considered as a spot of color in juxtaposition to other spots of color, all the knowledge of color and texture of the mosaicist and of the pointilist painter could be immed-

*Presented at the 30th Annual Convention American Concrete Institute, Toronto, Feb. 20-22, 1934.
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[1]JOURNAL American Concrete Inst., June, 1933, *Proceedings*, Vol. 29, p. 403.
[2]*Proceedings*, Amer. Concrete Inst., Vol. 16, p. 70.

iately applied to concrete. It would eliminate the necessity for a long period of experimentation. It would, if a technique could be devised, permit concrete to participate in the traditions of these older arts and transform it almost immediately into an acceptable architectural medium. We thought that a suitable technique must be one by which the particles of stone could be distributed and exposed on the surface of the concrete in a predetermined manner. Reasoning from the surface to the mass is a natural process. It is particularly so to an artist. We, therefore, thought that the desired end could be reached by making the concrete so that any section through it would have the character desired for the surface. Current work in the laboratories at home and abroad indicated that by carefully grading the aggregates, highly desirable qualities could be given to concrete, such as increased strength and density. Two methods of gradation were being studied. That by which the aggregate was evenly graded through many sizes from fine to coarse, and that by which it was graded into three sizes, fine, medium and coarse. Practically equal strengths and densities were obtained by either method but it always appeared that, when the latter method was applied to aggregates passing a half inch sieve, the best results were obtained by omitting the medium size. We, therefore, designed a two-step gradation which proved itself to be just what we wanted. It gave to concrete of the type in which we were interested the best structural qualities and characteristics of appearance adaptable to our theme and quite different from the appearance of concrete made with aggregate graded by other methods. Furthermore, our two-step method of gradation gave to concrete better workability than did the other methods. It prevented segregation and bridging and gave better flow. It permitted us to fill perfectly the most complicated molds.

Another example: We developed for the casting of Lorado Taft's Fountain of Time[3] a control of the water-cement ratio in the molds at the time of set by means of an absorptive core, which as part of the mold extracted free water and permitted the concrete to be placed in one consistency and to set in another. The Fountain of Time is so large a single group that we decided to cast it in place, in a plaster mold of more than four thousand pieces made on the original model. The usual process of casting is to pour material into a mold as into a basin, but in this case the usual process was reversed.

The material was packed between an inverted mold and an inner core. The core was framed with wood, covered with metal lath and a very porous plaster. This highly absorptive inner core drew off

[3]*Proceedings*, Amer. Concrete Inst., Vol. 19, p. 185.

the excess water, which had been used as a vehicle for placing the concrete, and left the concrete tightly packed between it and the mold in such a condition that it would not shrink away from the mold but would harden into a strong sharp cast. Here a major change in technique added something new to our concrete not only for the Fountain of Time but for succeeding work.

Again: For the Church of the Sacred Heart at Washington[4] we developed polychrome coloring by means of the aggregate to meet the requirements of the architects, Murphy and Olmsted, for a Byzantine-Romanesque church done in the manner of the churches of Ravenna. Technical control was exercised by means of raised contour lines in the molds. They permitted the use in one casting of many aggregates of as many colors, separated them and kept each in its own place without losing anything of unity in the mass of concrete. Design in color was now possible and concrete became a modern mosaic of unusual beauty with a character all its own and an adaptability greater than that of any medium with which we had had experience. I believe that this was the most impressive gesture ever made with architectural concrete. For us it was a great adventure. It stimulated us to efforts which can be clearly seen in a steady and rational improvement. Not for a long time did another problem force us to devise an essential change in technique.

In the years 1932 and 1933 we had entrusted to us two epoch marking jobs of work, namely: the ceilings of the passages to the courts in the new building at Washington for the United States Department of Justice and the dome of the Baha'i Temple at Wilmette, Illinois. Both of these works presented the difficulties, the challenge necessary to lift us above normal improvement to one of those extraordinary technical changes which give new and lasting character to a material. For the ceilings of the Department of Justice we devised a system of forming by which thin precast slabs of concrete mosaics were used as forms for structural elements and normal forming was eliminated. (With your permission I will reserve a discussion of these ceilings for another time.)

For the dome of the Baha'i Temple it was necessary to develop in the concrete greater early strength than we had done before. We did this by a new modification of technique. It retained a predetermined quantity of water in the concrete in the mold at the time of set by controlling the size, that is to say the surface, of the small aggregate.

The casts for the dome of the Baha'i Temple weighed as much as

[4] *Proceedings*, Amer. Concrete Inst., Vol. 20, p. 157.

three tons each. The nature of the molds in which they were cast made it necessary to turn them over within twenty hours and to remove the mold so that their surface might be treated to expose the aggregate. Frankly we were impressed. We felt the necessity for increased stability in these casts and we reasoned that it could be obtained by further decreasing the quantities of water in the concrete at the time of set. The difficulty was that we had been in the habit of extracting as much water as we could. We knew from experience that a properly designed capillary system when applied to wet concrete would extract all free water, that is, water which is not restrained in concrete by some force equal to or greater than the force of the capillaries. We also understood that this restraint is exercised principally by the surface of the aggregates and of the cement, to which water attaches itself with ever increasing tenacity as the particles become smaller. From this we reasoned that if surface could be brought under control a predetermined quantity of water, either more or less, could be retained in concrete against the pull of a capillary system. We learned that control can be exercised to a remarkable degree. Concrete can be designed from which water will run freely or in which water will be retained against the force of capillarity.

Our concrete is composed of materials generally grouped into three sizes, the large aggregate, the small aggregate and the cement. We applied our theory to the small aggregate because the surface of the large aggregate was insignificantly small and because the surface of the cement was too tightly covered by water to afford us much hope of success. Changes in the size of the small aggregate produced the exact result desired. We extracted the additional water, obtained the increased stability in the concrete, turned over the three-ton casts in twenty hours, removed the mold and exposed the aggregate. Exactly what we did was to increase the mean diameter of the small aggregate .0015 in. by changing the opening of a sieve from .0125 to .014 in. Considered casually it seems ridiculous that so small a change to but one of the ingredients should make so great a difference in the character of the concrete. It might be interesting to note that particles of the size indicated are about the largest which may be classified as small aggregate with surface sufficiently dominant to be subject to this technique. The application of this theory clearly proves that less water makes better concrete. But it should be remembered that water means practically nothing to a concrete product in its first phase. When concrete is being mixed and placed, water is only a vehicle carrying the solid particles. But water is of great importance in the second phase when the concrete is at rest in its mold and beginning to

set. Further, our experience teaches that the new technique will control not only the strength of concrete but its density. There is no need for elaborate tests to establish this. It is perfectly apparent to one watching the concrete in the mold while the water is being extracted, and to one handling and studying the casts after they have been made.

Here, I believe, is another extraordinary technical improvement devised for the execution of the dome of the Baha'i Temple, which gave an added character not only for the improvement of architectural but of structural concrete. Here is a means to control over a wide range the water-cement ratio in concrete at the time of set. It is a tool from which much is to be expected. We believe that such close control of water will be of ever increasing value as thin sections come into more general use. Indeed it may be that thin sections will never come into general use without such a close control of water.

Other interesting processes and devices were used in casting and erecting the temple dome. Some of them are new to concrete construction and others of them are improvements. The character of the work was such that one major technical development was not sufficient to meet all requirements. Many minor improvements and ingenious devices were also needed.

The process of exposing aggregate evenly over the surface by brushing concrete with wire brushes before it is thoroughly set is still essentially of the technique of making architectural concrete of the exposed aggregate type and is now used in our studio for all our work with concrete. Certain difficulties imposed by the process would be relieved if a substitute process could be found to expose the aggregate properly after the concrete had set. Our experience with other methods such as rubbing, mechanical brushing, chemical treatment, sand blasting, tooling and the like have not been satisfactory. Hand brushing establishes beautiful architectural planes, uniform surfaces and good drawing. The other methods have produced for us poorly established architectural planes with marked erosion and bad drawing. The advantages of hand surfacing are still more apparent when colored aggregates are used. More violent methods fracture the surface of these aggregates and change their color.

The mold or forms in which concrete is cast are at present one of the great difficulties of the industry. Complicated forms test the skill of a craftsman and are a handicap on the performance of concrete in

the architectural field. Had the molds for the Baha'i Temple dome been necessarily made with some nonplastic material, as wood or metal, the difficulties and the cost might have endangered the project or might have defeated it. Indeed there were many who admired the beauty of the temple dome but who thought its execution to be impossible or impractical. Fortunately we had had much experience in making molds for other unusual projects. We knew the remarkable adaptability and economy of molds made with a plastic material when applied to complex forms, therefore: we made the molds of the temple dome with plaster. Although these molds were far more complicated than any we had previously made for concrete, they added nothing new in principle but much to experience.

It was difficult to fill the deep and narrow molds of the great ribs. It was particularly difficult to fill the molds of the perforated sections of the ribs. They were channel shaped and consisted of an inner and outer form with five inches between. This space was almost completely choked by projections designed to form the holes in the ornament and by reinforcements. Here, truly, was a need for two consistencies. One for placing concrete in these complicated and unhandy molds and one for a strength to meet previously explained requirements. The problem presented was paradoxical. If the consistency were wet enough to permit the concrete to fill properly such a mold the required strength would not be developed and if the consistency were dry enough to develop the required strength the concrete could not be properly filled into such a mold.

The casts were cured in a chamber in which the air was kept close to maximum humidity by intermittent spraying of the floor. The floor was covered to a depth of about three inches with pebbles screened through a one quarter inch sieve. Evaporation from the floor kept the air moist but not filled with spray. The intention was to keep the casts from drying out and not to add more water.

The structural steel designed to support the concrete dome was composed of curved ribs radially spaced and of straight purlins fastened on top of them. This structure did not coincide with a spherical form well enough to support the ornamental envelope. We, therefore, imposed upon it a furring system of light steel tees bent to conform to a sphere and to the underside of the envelope. These tees afforded a support along two sides of each piece of concrete and a means to fasten the concrete envelope to the structural steel. The concrete dome was divided into three hundred eighty seven pieces, which corresponded to the panels formed in the structural steel by the intersecting ribs and purlins.

At every corner of the concrete casts there was inserted a steel fitting drilled and threaded to receive a cap screw. The inserts were placed so that each fitting in the top of a cast could be joined to a corresponding fitting in the adjacent cast by bolting to them a steel plate passed behind the furring tee. This arrangement held each concrete cast close to the furring tees but permitted it to move as might be required by expansion and contraction in either the concrete or the steel. To the fittings at the bottom of the casts were attached short pieces of steel angles, which rested on the purlins and prevented the casts from slipping down. When the concrete casts were set by this method there was an open joint one half inch wide on every side, and each piece of the concrete dome was completely free from every other piece. I have no knowledge of another masonry structure assembled by this method. It is a logical one even though somewhat contrary to precedent.

The members of the Baha'i Faith look upon their Temple as a building which will last for a long time and so does this studio. Every precaution has been taken to make the concrete as well as it can be made in the present state of the art. We believe that the concrete in the Baha'i Temple will endure better than terra-cotta, freestone, marble or any other building stone excepting granite. At the same time our studio, because of its experience with all these materials, knows that masonry materials in architectural form will not endure indefinitely. To monumental buildings which have stood for a long time, as time is reckoned in human history, there have been many repairs and replacements. We have, therefore, arranged the temple dome so that any piece can be repaired and, if need be, removed and replaced without disturbing any other piece. Further, if the furring system through neglect should deteriorate to such a condition that it were advisable to replace it, it can be moved and replaced without disassembling the concrete dome. These provisions for maintenance should be regarded neither as unnecessary precautions nor as a lack of faith in the durability of any of the materials. Indeed it would be presumptuous to attribute to the steel structure and the concrete envelope an endurance greater than they can possibly possess.

Materials were chosen with care like to that exercised in making and assembling the dome. White quartz was selected for aggregate because it is beautiful and strong and can resist erosion and corrosion. Copper bearing steel was used for the furring system because some metallurgists say that steel containing a small quantity of copper will not rust as readily as plain steel. We were not greatly impressed

by this but if the effect be there we wanted the dome to benefit by it. Chrome-nickel-steel alloy, usually called stainless steel, was chosen for the fittings which hold the concrete casts in place. It can hardly be called stainless but certainly it has shown a good resistance to rusting, as the term is generally understood. We felt the need of a rust resisting metal for the fittings because, if rust worked back between them and the concrete, it might break the corners of the casts. We considered aluminum but thought it might corrode excessively in concrete. We disliked the green stain of bronze. From other suitable metal we selected the steel alloy as the best obtainable within reasonable costs. The fittings have since been inserted in wet concrete, stored in damp storage, washed with muriatic acid and weathered, all in the process of making the concrete dome. We are pleased by their performance and feel assured that their deterioration will be very slow.

I understand from the engineer in charge that the economy afforded by concrete for the ornamental dome of the Baha'i Temple was truly remarkable. Great difference between the cost of concrete and that of other material is to be expected when the work is difficult and complicated. When the work is simple the difference in costs is not so great. But, when concrete is properly used, when the technique is intelligent, there is always economy, freedom of design and a flexibility unequaled by another material.

Let the Baha'i Temple be admitted to evidence to support my testimony that concrete of the exposed aggregate type is no longer in an experimental state but is ready for use and is an entirely satisfactory architectural medium. All present indications point to exposed aggregate as the mark of architectural concrete. I see nothing in the art of making concrete which threatens its supremacy. I know of no existing process which is likely to set up another element of concrete in the place of the aggregate to dominate its appearance, and I repeat what I said more than ten years ago, namely: the aggregate is the dominant element of concrete, therefore, the appearance of the concrete should be the appearance of the aggregate. Further, it has been thoroughly demonstrated that the character of the aggregate has been made to control the character of the concrete and that such concrete has been made to meet every architectural requirement. I do not hesitate to assert without weakening qualifications of any kind that from the point of view of designing architect, artist architect, and the studio executing their work there is no masonry material with which as much of form and color can be expressed as with exposed aggregate

concrete. That is a positive statement. It is as definite as any statement I have made before the Institute. I mean it. I can support it. The reason for making it is that it is now time to make an end of unbelief and doubt in concrete as an architectural material. The architects who still doubt are depriving themselves of a great and efficient medium, with which to solve modern architectural problems. They might do well to investigate and to learn why Louis Bourgeois chose exposed aggregate concrete for the execution of his exotically beautiful temple and why Zantzinger, Borie and Medary chose it for the strikingly colorful ceilings of their Department of Justice.

The presentation of the foregoing paper by Mr. Earley was followed by steropticon views with descriptions of details of manufacturing and construction methods, devices and procedure. Their presentation here is in turn followed (p. 274-278) by "convention discussion"—consisting almost wholly in questions from the convention audience and Mr. Earley's answers.—Editor

The Baha'i are a Persian faith which originated about 70 years ago and came to this country by way of the Paris Exposition of 1900. The architect, Louis Bourgeois, (deceased), had, as the dream of his life, that he might execute an architectural symbol of a new religion which would not be reminiscent of the forms which had served as symbols for other religions. The sketch indicates the nature of the problem for execution in architectural concrete. The dome is about 100 feet in diameter; the decorations perforated. Reduced in scale, they are as fine as a piece of Duchesse lace, and these perforations are carried down through the building in grilles filling all the openings and with architectural surfaces covered with a tracery as fine as is found in the marble slabs of the Taj Mahal.

Our studio had nothing to do with the construction of this skeleton (described by Benjamin Shapiro in a paper before this Institute*). The dome is of glass to serve as a watershed. Above it is now superimposed the perforated concrete dome. Its theme is that by day the light of the sun will filter into the Temple, symbolic of the light of faith and at night the light of the Temple will filter out to illuminate darkened world. It is called "A Temple of Light." It is of very unusual design. The plan of the first lift is of a nine-pointed star; the second is the same, but oriented so that the points do not coincide. First a record had to be made of the physical condition of the dome. Men made wooden templates on every line of pylons around the dome, because to pre-cast the dome sections in concrete in our studio in Roslyn, Va., and then to ship and assemble them at Wilmette, it was very necessary that no mistakes be made. The dome was very complicated by reason of the fact that the ribs were not radial. It would have been difficult to translate the necessary calculations into the mind of the craftsmen who had to do the work.

*Journal, Amer. Concrete Inst., Jan.-Feb. 1934, *Proceedings* Vol. 30, p. 239.

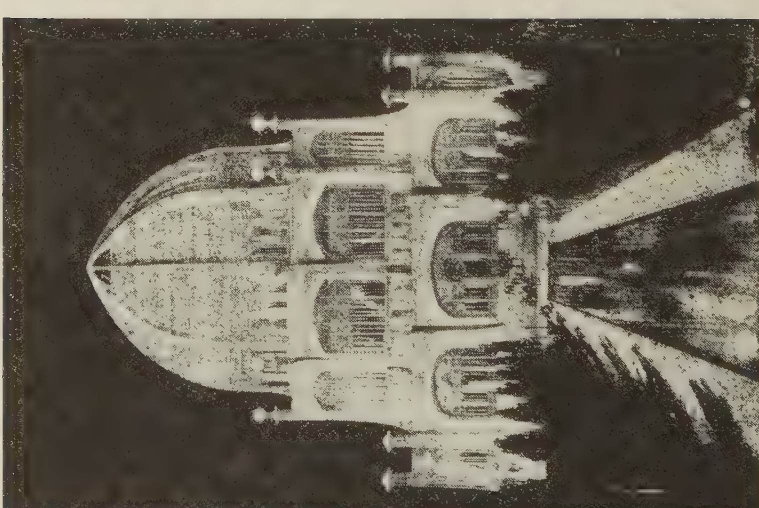

FIG. 1—FROM A SKETCH OF THE BAHA'I TEMPLE BY THE ARCHITECT, LOUIS BOURGEOIS

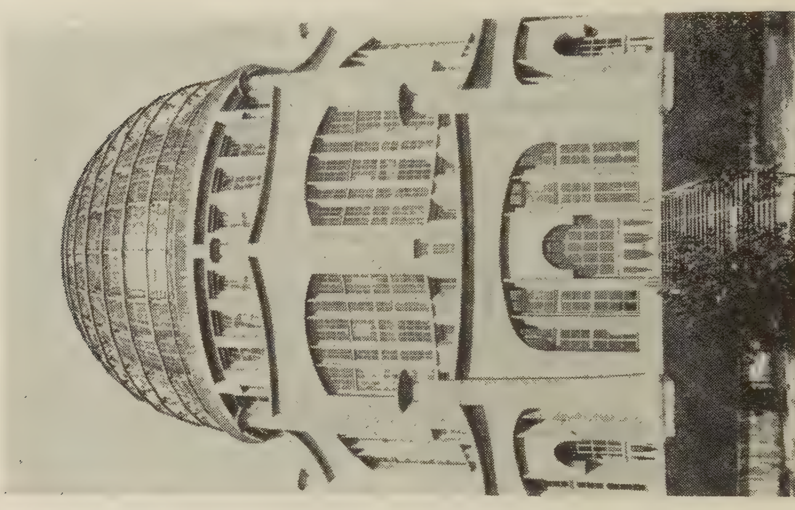

FIG. 2—THE STEEL AND REINFORCED CONCRETE SKELETON OF THE TEMPLE AT WILMETTE, ILLINOIS–ON SHERIDAN ROAD WHERE IT TURNS TOWARD LAKE MICHIGAN

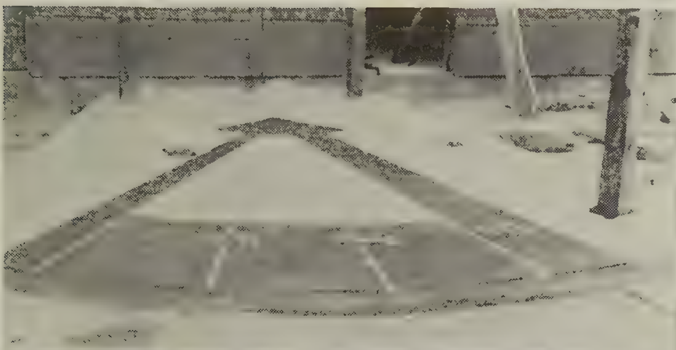

Fig. 3 and 4.—Surveys marked on concrete bases for a full size model of one ninth of the dome

We decided to build a full size model of one-ninth of this dome so that all the measurements and lines could be taken off it in a series of templates rather than in a series of calculations. First, on a concrete platform, a full size projected plan of the dome was laid out. The white lines (Fig. 3) indicate the joints between the sections of the field and two ribs. From the periphery of the dome another plan was projected of the outer edge of the dome. The dark spots indicate sections through the rib and the dark line connecting them is the plan of the five-inch thick concrete envelope of the dome. On top of this full sized plan of a ninth of the dome we constructed a scaffolding; timbers were set with exact relation to the steel which existed on the dome.

FIG. 5, 6 AND 7.—A SCAFFOLD SUPPORTS TIMBERS CONSTITUTING A REPLICA OF THE STEEL OF A DOME SECTION

Over timbers exactly representing the steel work were timbers exactly representing the purlins and over the purlins in turn were placed strips representing the thickness of the concrete field sections of the dome, (Fig. 5 top left). Both ribs and purlins were marked with center lines (Fig. 6 below). The great ribs were laid out in plan on the ground in relation to a vertical section through the dome. The framework running up over the section describes one of the great ribs (Fig. 7, top right).

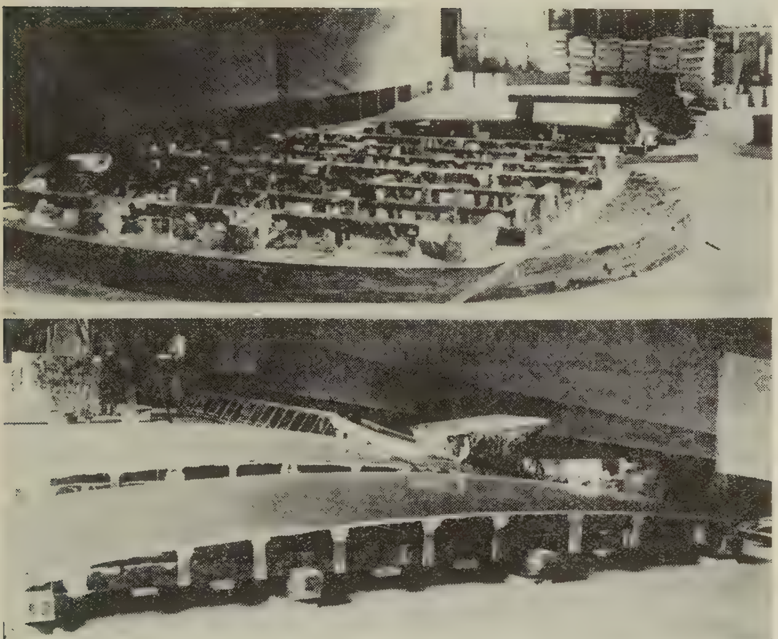

FIG. 8 AND 9.—FIRST STEPS IN MAKING MODELS

After the layout had been made it was necessary that the configuration of the dome be translated from the model into the workshop as a basis upon which to build the models of the field of the dome. That was done by spinning on the floor a plaster disk like a saucer. (Fig. 8, top). Then there was laid off on that the lines which correspond with the boundary lines of the field of the dome. On the saucer-like form plaster slabs were cast. The timbers are merely reinforcements for the back of the slabs, and those slabs were taken off this saucer-like shape and placed on the floor at some other position. Subsequently they were sawed into lengths and put together to give sections of the surface of the dome as might be needed for the work. (Fig. 9, bottom). Such curved slabs formed the basis of all models of the dome.

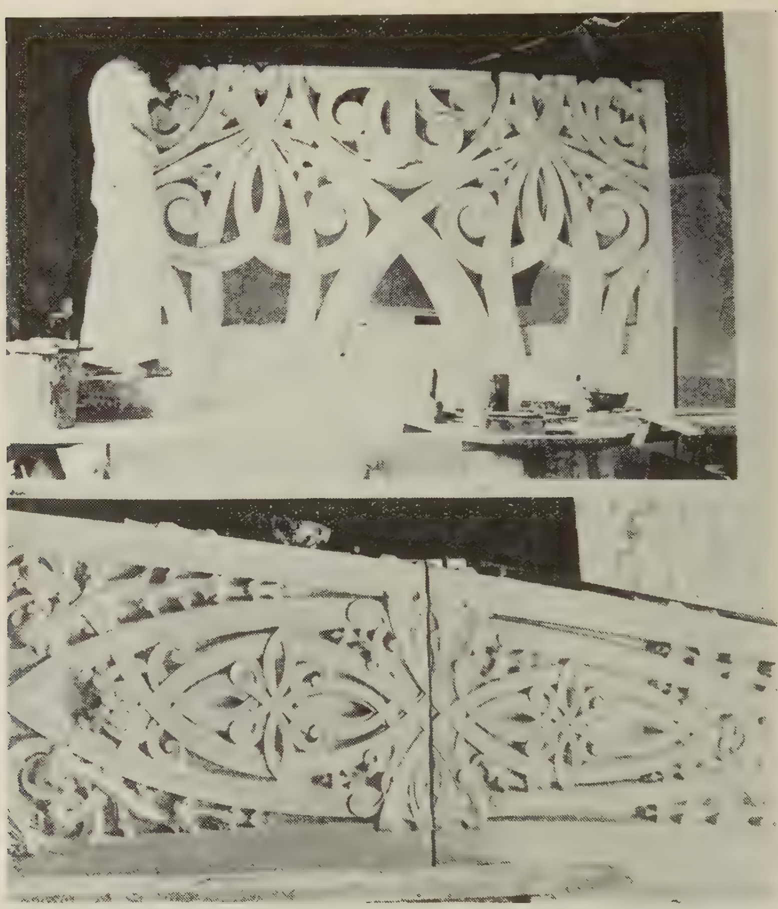

FIG. 10 AND 11.—DETAILS WERE CARVED IN PLASTER MODELS

Models were first roughed out in clay and then cast in plaster and then re-carved by hand. (Fig. 10, top). This was done so that the lines, the drawings, of this very complicated ornament might be as true and as nice as possible. Then again it was done so that the modelling on the face of the dome might be carefully done with due consideration to this phase of the problem: If the modelling of this ornament were overdone by erecting projections or by excessive perforations, the continuity of the architectural dome would be lost. If, on the other hand, the surface of these models was without movement, the dome would present an appearance more like a colander, just a plain surface with holes punched in it, which would not be architecturally acceptable. Fig. 11 shows two sections of the great ribs placed together on the pattern of the dome showing the half inch joints which separated them.

FIG. 12 AND 13.—PLASTER MODELS ERECTED ON THE DOME FRAME

Models were carefully checked by placing them on the frame of the dome. (Fig. 12 13).

The bottom sections of the great ribs extending down over the clerestory were not a part of the dome: they were modelled separately. (Fig. 14, 15, next page). The top of the model coincides with the springline of the dome. In this assembly it was possible to judge the character of the ornament and the character of the modelling to be sure that the uniformity of surface was retained and the proper degree of decoration and that there had been achieved a proper balance of perforations with the general area of the dome. Note the half inch joint surrounding every piece—each casting independent of every other. The bottom castings were about ten feet square, five inches thick and as finally cast of concrete weighed between three and three and a half tons each.

FIG. 14 AND 15.—BOTTOM SECTIONS OF RIBS TO EXTEND DOWN OVER THE CLERESTORY

After the models were made, the next thing was to make molds on them, and these molds were necessarily very complicated (Fig. 16, 17, 18) because the ornament is perforated, which means that wherever there is a perforation in the ornament, there must be a projection in the mold. Those projections were five inches high and very numerous, which meant that if a concrete casting were made in a mold as ordinarily constructed, it would be impossible ever to remove that mold, because if, in moving it, it was twisted the slightest bit, all of these projections would bind so on the perforations that it would be entirely impossible to remove the mold. Another thing, these molds had to be removed within 24 hours, before the cast was as hard as it would have to get. So, wherever there was a perforation, it was treated in the mold as a plug, and when the mold was removed from the cast, the plug would remain in the cast and it was removed separately, and afterwards re-assembled in the mold.

FIG. 16, 17 AND 18.—PLASTER MOLDS WERE MADE FROM THE MODELS

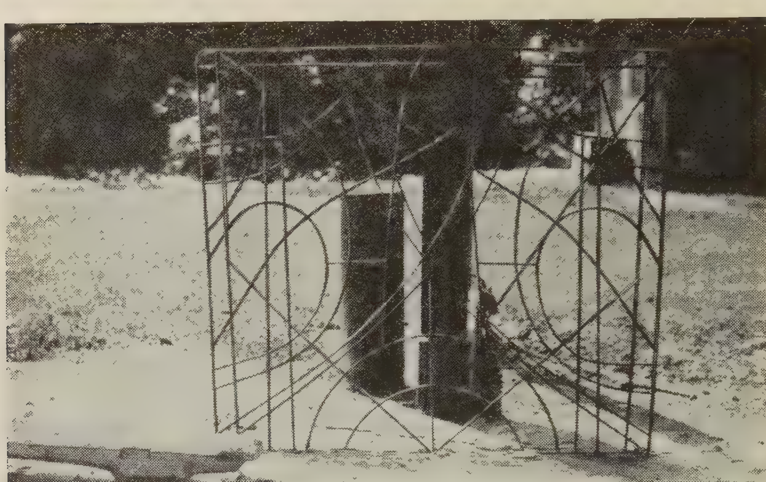

FIG. 19—A REINFORCEMENT UNIT

Reinforcement (Fig. 19) was designed on the theory that if sections of the dome could be held rigidly around the edges, there was very little likelihood that castings would flatten. That is a thing we have to be very careful about, because slab castings, particularly when they are new, have a tendency to bend; if they are flat they will curve and if curved they have a tendency to flatten. Every one of these castings is a section of the curved surface of the dome and we held the edges of them as firmly as we could. Reinforcements were bent to follow the curvature of the ornaments and wherever they crossed they were electrically welded so that each reinforcement unit was a welded mesh.

It was the desire of the architect that the dome should be the whitest thing possible and we have learned that so much white presents a difficult problem. Because we thought that, even though the surface was to be broken by ornamentations and perforations, there was great danger that it might be dead or chalky or have the appearance of a plaster casting, we chose an aggregate to give a maximum of reflection —a white crystalline quartz. Studies made with the white crystalline quartz, while they were better than dead white surfaces, because the broken faces reflected light, showed too little scintillation to avoid monotony. So, we also chose a clear translucent quartz. We mixed about one-quarter of the translucent quartz with three-quarters of the white opaque quartz and the result was very pleasing. This quartz came from South Carolina and the clear quartz from a little deposit near Lynchburg, Va. Because our requirements for size are so exact that nobody has any sympathy with us, we crush our own aggregates. The quartz passes through a jaw crusher to an elevator, through screens and back through a set of balanced rolls, and it keeps circulating, and whenever the stone passes through one of these screens it passes to its proper bin. By this method we effect an economy because the amount of crushed material of one size which may be expected from the mass of raw material is about fifteen per cent. That was impossible, because some of the aggregates we have used, (some highly colored ones) cost as much as $2,000.00 a ton and fifteen per cent

FIG. 20 AND 21.—CASTING OPERATIONS

is not a satisfactory recovery in usuable product. Therefore by taking out all particles which are the right size, between every crushing operation, so that further attrition does not further reduce those sizes and by using two sizes in a two-step gradation, we are able to get 70 per cent of usable product out of the crude material.

Pictures will not show, so you must accept my statement that all particles of each of the two sizes we use are as nearly of one size as it is practical to make them. To indicate the character of the screening, I would say that the size is such as you might expect to have between alternate sieves in a set of standard sieves.

We used a little open mixer, for a one bag batch. We find that by using a small mixer that is open, in which we can see the concrete while it is being mixed, we can vary the consistency as the cast progresses to meet our requirements. I suppose that sort of thing applies to our own particular work much more than it would to ordinary concrete.

Casting this dome involved matters of economy as well as artistic problems: When the molds were finished from models done with the greatest care, our mental attitude changed and we made an effort to produce casts with the least possible

effort that would maintain quality. We have a shed which is covered by a light framework; in this molds were set on concrete foundations in a crane line one after another, and every day every alternate mold was filled (Fig 20, 21). Across every mold an angle iron was bolted down to a fitting in the mold. The angle iron has holes bored in it to serve as a jig in placing the fittings (used in final assembly of the dome) which were bolted to the angle (Fig. 21). The iron served also as a gage for centering the reinforcing web in the concrete casting.

FIG. 22 AND 23. BRUSHING THE SURFACE TO EXPOSE THE AGGREGATE

The following morning these castings were turned out from the mold and leaned in a vertical position against posts. That put them in exactly the right position to be brushed and surfaced to expose the aggregate. Some may wonder that so many

men are employed on one piece. The reason is that concrete which has stiffened enough in 18 or 20 hours to permit its removal from molds and to stand it up, has a tendency to keep on hardening, and it is very wise to finish the surfacing as soon as it possibly can be done, because the difference in the hardness of the surface between morning and afternoon is a thing you would have to experience to believe.

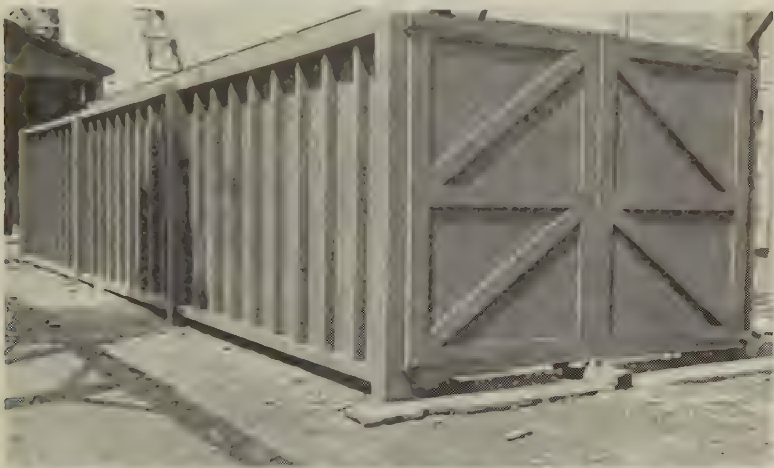

FIG. 24 AND 25.—CURING CHAMBER

While the casts are being surfaced the alternate molds are being filled and the molds released are reassembled for their next pouring the following day. When casts have been washed they are picked up by the crane and stored in a damp chamber (Fig. 24, 25). This has wooden walls, plastered inside with emulsified asphalt and a roof of canvass in panels on light frames easily lifted off when at the end of two weeks curing the crane lifts casting out for air curing and shipment.

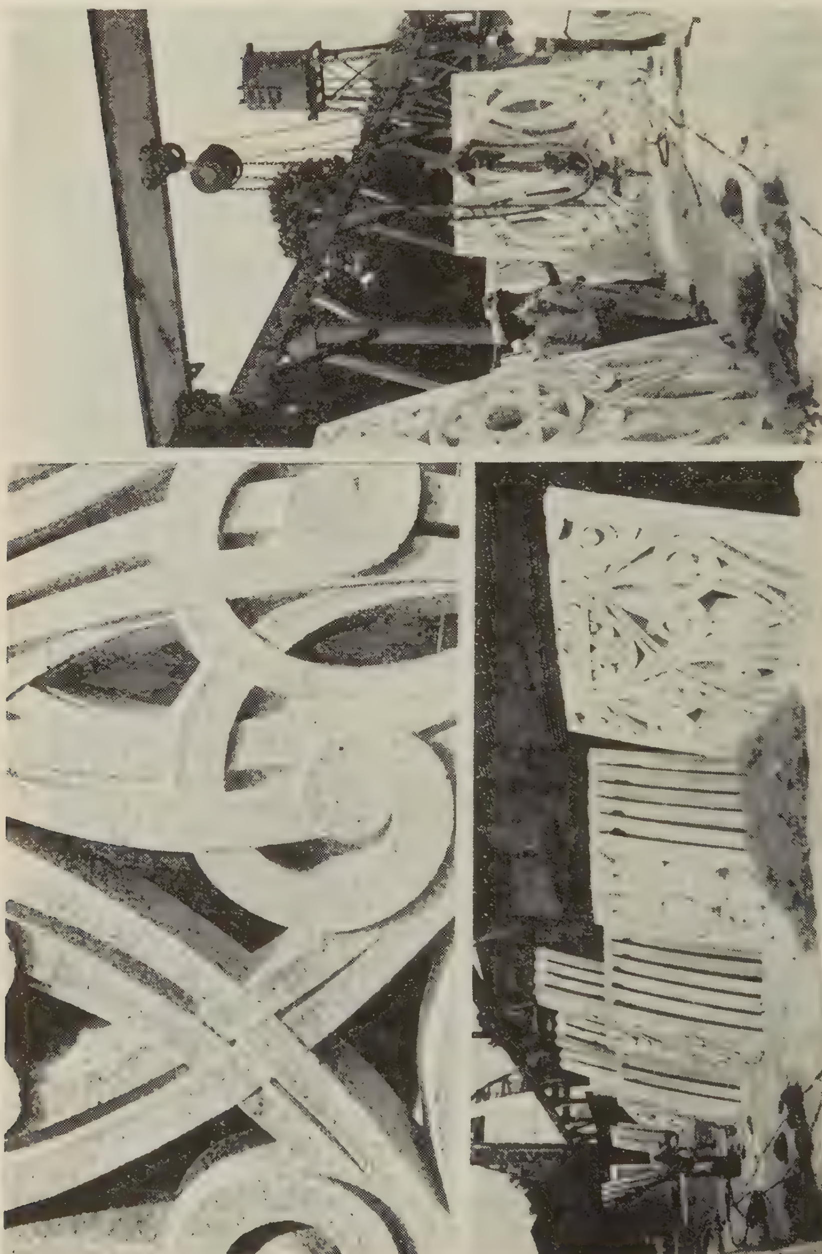

FIG. 26, 27 AND 28.—CASTINGS READY FOR SHIPMENT

Fig. 26 and 27 show castings of dome sections preliminary to shipment. Fig. 28 indicates about the scale of the texture and also the varying shades in the color of the two kinds of aggregate. The darker spots are of the translucent quartz and the white spots are the opaque quartz. Though the clear quartz appears dark in a photograph it is, as seen in the casting, not a white spot but a bright spot. We determined in these castings that we would avoid patching, and wherever something occurred that was a defect of an inconsiderable character, we just frankly left it; it is better to leave them than to make an attempt at patching.

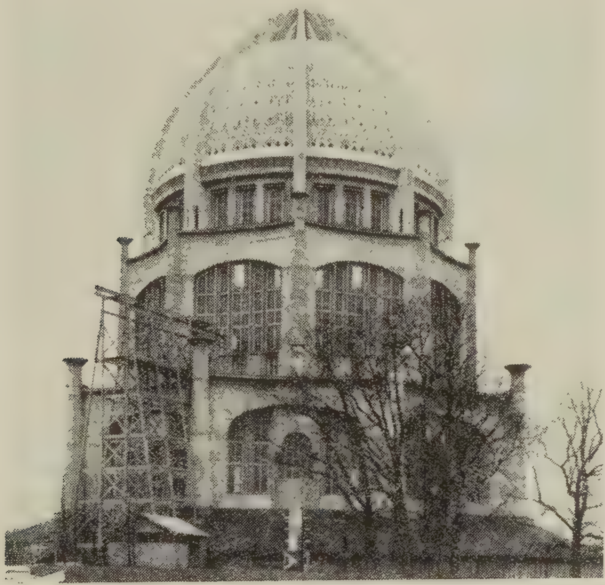

FIG. 29.—THE DOME AS NOW ASSEMBLED—THE REMAINDER OF THE STRUCTURE STILL TO BE COMPLETED

The drawings which were left to us by the architect from this point onward are rather sketchy. They are all that is necessary to convey his idea, but they do not in any sense express any of the details of the ornamentation. We feel however that the building below the dome is going to introduce an exceedingly interesting problem in ornamentation. We have already established in the dome a pattern, but as we come down we have two characters of surface—openings to be covered by perforated grilles which must be done in relation to perforated grilles in the story above, which in turn are related to the dome. So, while the dome and all the openings require perforations, we have structural surfaces which are not perforated and we are looking forward with a great deal of pleasure to the experience of relating this continuous ornament so that we will not lose the structure of the building nor the sense that the structure is solid and that the openings are perforated ornamentations.

(See "convention discussion" of this paper on next page)

CONVENTION DISCUSSION

C. W. Morssen (Consulting Engineer, Montreal): I should like to ask which would have been costlier to cast the dome and carve it to obtain the perforations and ornamentations or to do the work just as it was done? Then, I should like to ask how many panels were wasted? Did you have any failures? The work is very complicated.

Mr. Earley: I haven't the figures which relate this dome in concrete to a dome in carved stone, but I can relate it for you in this way: A. B. McDaniel, the engineer in charge for the trustees, took estimates on this dome in stone, in cast aluminum and in terra cotta. The cast aluminum was subsequently withdrawn; the terra cotta was rejected because the trustees felt that the number of pieces involved would superimpose over this dome a pattern of joints which eventually would be the dominant pattern of the dome and they did not want that to happen. In stone the estimate was ten times the cost of the dome in concrete. It is a 100-ft dome and we will execute such a dome for $150,000.00.

Mr. Morssen: I did not mean a dome in stone, but a dome cast of concrete using the same aggregates, but with carved ornamentation.

Mr. Earley: You could not do that. Having selected quartz as the aggregate, the concrete is strong and the quartz exceedingly hard and steel tools, such as the drills and the points, would not be hard enough; after a few blows the points would be gone. If it were desirable to carve such work, it would be much better to use another softer aggregate. Now, as regards the cost, however, I can say this, if it will help to relate if for you: the largest castings are about ten feet square, and the smallest one at the top of the dome, about three by ten feet average dimensions. One of those castings can be made by two men in a day. You yourself know that you would not go far with that labor in carving. The other question was how many castings I lost. Oh, yes. Having decided that we were not going to patch this dome (although an excellent job in patching can be done in concrete by skilled mechanics if done at the proper time) I am proud to say not only on my behalf but on behalf of the shop, that we did not lose a casting. (Applause)

Arthur R. Lord, (Chicago): I should like to ask Mr. Earley what the water-cement ratio was and what the strength of the concrete at some time might be?

Mr. Earley: As you have asked the question, Mr. Lord, I cannot answer it, I will have to answer it relatively. You must remember that in the studio we are not scientists, we are craftsmen, and therefore we may be permitted to have a little different mode of expression and think of some things in a different way. In making concrete of this type, we do know this, that we can cast a Corinthian capital maybe six or seven feet across and take the molds off in a day, and we know that you cannot do it with some other types of concrete. As far as the water-cement ratios in the cast are concerned, you might judge of that in this way; the concrete was mixed until it worked well, it had about five gallons of water to a sack of cement. At each of those little one bag mixers, they have a quart milk bottle with which to add a little more water as judgment dictates. Successive batches for a single cast usually require less water. After the cast is filled, then comes the second water-cement ratio; water is extracted from the concrete in the cast until it is impossible to pull any more out of it, and when it is in that condition, you cannot make an impression in the concrete by leaning on it; you can stand on it and cannot make an impression on it. I would make a guess that the water is less than four gallons to a sack of cement. I am sorry I cannot give figures in the terms you would like.

Mr. Morssen: I should like to ask another question. Do you ever use vibrating tables?

Mr. Earley: Whenever I speak of those things, I always have to speak of them in relation to our own type of concrete. As you all know, there are many types of concrete for different purposes and we have tried all types of vibration, varying in frequency, and we find that, for our purpose, the best type of vibration is one that is a jar. The aggregate in our concrete is in just two sizes. The volume mixed fills what you would expect it to fill; we do not have a "yield" at all. The purpose is to keep the large aggregates in contact with each other as far as it is possible to do so. There being but one size of large aggregate, the particles are in that relation you would expect to find in a receptacle filled with marbles. If you shake the receptacle at high speed that does not seem to do our concrete very much good. After we have filled a mold with concrete, a column mold, for instance, we pick up a piece of three-by-four and crack that mold on the side a good sharp crack, jolt it, and we find that that settles the material down. You can see it settling in the mold. Another thing you must remember is that with concrete of this type in which all the particles are of practically the same size, the aggregate has the characteristic of

sands in an hour glass. All the sands in an hour glass are about the same size and the material flows through and it does not bridge. If you go around on the side of our sand bin and make a nail hole in it, it will empty the bin; all the sand will come out, and in my paper I called attention to the fact that this gradation gives unusual workability to the concrete, so that in filling the molds a jolt will set this concrete down into every corner and every particle of aggregate down between the other particles and I will go still further—it will turn the aggregate on the bottom over so that the flat side lies up against the mold.

Secretary Whipple: I think there are several in the audience who would be interested in having you tell them how you apply to these castings the capillary system which pulls out the excess water.

Mr. Earley: It is just too simple. The first time we ever used the capillary system to pull the water out of the concrete was many years ago, when we were doing some balusters at Meridian Hill Park in Washington. Those balusters were designed by Cass Gilbert; when he drew the baluster, he drew it from the top to a certain point and from the bottom to a certain point, and then he was through. It was a beautiful baluster, but it was an exceedingly difficult form to cast because the bowl is large and the base is large, but the necking at the top and at the bottom is very thin. In that way he gets a vigor in the baluster and a decoration at these thin points. When we were casting these balusters we put five balusters in the molds every day and took them out and threw them on the dump the next morning. We cast a great many that way. It cost us $1500.00 to make the first baluster. The reason was that shrinkage in the concrete left an incipient crack around the neck of every baluster. We could have pointed them up and sent them out, but it wouldn't have been sporty. Finally, we decided that the movement was due to the water in the concrete. We filled those balusters and piled the concrete up on top of them and shrunk them down; then we took a piece of newspaper and spread that across the top of this wet concrete. Of course the newspaper acts like a piece of blotting paper and starts pulling the water and it was not very long before the newspaper was wet all through, so in order to continue to give volume to the newspaper we piled a very fine sand on top (the refuse from our crushing system) and that pulled the water up out of the balusters so that the concrete was stiff as I described it to you. Those castings came out all right—not like beads on the reinforcement. We continued generally to use that system which consists of something that will act like a piece of blotting paper, maybe

a piece of newspaper or cotton cloth or anything of that kind, and then piling very fine sand on top of it to give it the necessary volume.

P. H. Bates (Washington, D. C.): Earlier this evening a speaker showed a picture of the peristyle of the Parthenon, at Nashville but intimated he did not know how that work was done. Can you tell us?

Mr. Earley: The Parthenon at Nashville was of stucco, built as part of a group to commemorate the centennary of Nashville's admission to the union, and used as an art gallery. It was made fairly closely in accord with the Athenian Parthenon. The building was unusually pleasing, and the city of Nashville decided to make it permanent. Very soon the stucco deteriorated so it had to be replaced and finally they came to us to do it in concrete. Now first of all, because it was an art gallery, housing many loaned pictures, it was necessary that the building have some degree of fire resistance. The walls of the body of the building were of brick, two eight inch walls spaced about four inches, and they, in turn, were built on rubble stone masonry. When the restoration was made it was decided to keep the old brick walls and finish them with the same type of concrete used to cast the columns, etc. The way we did that was this: The walls are about 34 feet high and nearly 200 feet long on the long side of the building. Among the most difficult problems is to do a very large wall surface pleasingly. Sometimes a wall surface such as that is much more difficult as an artistic problem than the dome of Baha'i Temple, because there is so little you can do and it must be done in good taste; so we decided then that it would have to be divided up to make it easier to look at, and we took for the dimensions the courses of stone in the Athenian Parthenon. We applied an undercoat of portland cement and sand stucco to the brick walls, which brought them all out nice and even. Then we stripped the walls with wooden strips, horizontally. We next applied the same materials used to cast the columns, but applied it in alternate strips all the way up the building. When the wooden strips were taken down, the courses served as forms placed for the alternate courses. We trowelled one course one way and each adjacent course the other way, and in brushing them out revealed the stones a little deeper on one than the other. There were vertical joints in the stones of the Parthenon but we felt we had no right to put them there because they would be frankly an imitation of the stone courses, but we had a reason which justified us in banding the building in the way we did. That brings us to the hand brushing to expose the aggregate. There are many things that enter into our sensation of surfaces, of which in a new structure you cannot properly avail yourself.

The work you admire in an old chateau has a character, a patina on the surface, that is the result of irregular erosion in the different places, and you would not want to errode the surface of a new building by some artificial method which would not be frank or necessary. But to avoid large monotonous areas there is no reason why we should not brush one area deeper than another, however, and profit by the play of lights and shadows over the surface.

For such discussion of this paper as may develop, readers are referred to the JOURNAL *for October, 1934 (Proceedings, Vol. 31). Discussions should be available to the Secretary by August 1, 1934.*

Manufacturing Concrete During Cold Weather*

BY R. B. YOUNG† AND WILFRID SCHNARR†
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Much has been written on the economics of winter construction, and on the precautions to be followed if concrete work is to be done in cold weather, but little has been published as to just how much heat needs to be supplied to concrete under different conditions and how that heat can be most advantageously introduced into the mix. In this paper, the authors have attempted to discuss these problems in the hope that their experience may be of help to others who, like themselves, must manufacture concrete in cold weather.

Obviously, the amount of heat to be supplied to a concrete mixture will depend on the temperature which that concrete must have at the time of placing and the initial temperature of its constituent materials —the cement, aggregates and water; and the answers to the questions of the previous paragraph will be only found after determining first, the temperature which the concrete should have in the forms and second, the amount of heat which the concrete will lose in handling.

TEMPERATURE AT THE FORMS

The critical temperature of the concrete in the forms is that existing at the time when protection begins and any additional temperature that it may have, when placed, is primarily to compensate heat losses sustained between the times of placement and protection. It is difficult to estimate with any accuracy the amount of these heat losses, for they depend on many factors, the outdoor temperatures, wind conditions, type of forms, mass of the concrete and degree of protection afforded the concrete during the interval of placing. A safe rule to follow in ordinary building construction and other work where the structural units are light, say of less than 2-ft. minimum section, is not to permit the temperature of any concrete in the forms to drop below 50° F., while the average temperature should range between 60° and 70° F.

*1st of two papers by these authors on cold weather concreting presented at the 30th Annual Convention, American Concrete Institute, Toronto, Feb. 20-22, 1934.

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Where the concrete is placed in heavier sections, a temperature of 60° to 70° F. is not necessary, especially where the concrete is in large masses, and here its minimum temperature may be allowed to drop even to 40° F. The heavier masses will generate considerable heat, but it takes about 24 hours for this heat to begin to raise the temperature of the concrete to any extent so that some protection of the surfaces is immediately necessary. After about 24 hours the average temperature of the concrete will rapidly exceed that of the thinner sections with heat applied, and this heat can be utilized to some extent for curing.

Here a word of caution lest someone thoughtlessly misinterpret the authors' statements. While the critical temperature of the concrete in the forms is that existing at the time protection begins, this critical temperature is not the average temperature of the concrete but its minimum temperature or the temperature of its coldest part, which is that portion most exposed to cooling. In thin sections, there will be practically no difference between the average temperature of the concrete in the form and its minimum, but in mass construction the concrete adjacent to the forms may be much colder than in the interior, especially if the outside temperatures are sub-zero. In the latter case, the mass of the concrete, although warm, will not transfer heat to the surface fast enough to offset the cold penetrating the forms. It will be necessary to protect the more exposed surfaces even while the concrete is being placed.

It is very important though often overlooked, that since concrete has to be protected in any case, it is advisable to install the protection prior to concreting, not after the concrete is in place. In sub-zero weather this is not only advisable, it is absolutely necessary if the concrete is not to be damaged, for at 0° F. or lower no amount of heat available in the concrete by virtue of heating its ingredients will protect its exposed surfaces from frost. It is well to remember, in dealing with the protection and curing of concrete, that anything which weakens a surface, subject either to wear or weathering, is to be avoided, for here, if anywhere, the motto of the paint industry holds: "Save the surface and you save all."

TEMPERATURE OF CONCRETE WHEN DELIVERED TO THE FORMS

The temperature at which concrete should be delivered to the forms depends not only on the lowest permissible temperature for the concrete in the forms, but also on the loss in heat during placing and until artificial heat can be supplied by protection. Experience shows that concrete loses relatively little heat in handling and where, as is often the case, large masses of concrete are placed in protected areas, the

heat losses are negligible. In lighter construction, such as reinforced frames, the heat lost by the concrete after being deposited, is considerable and the initial temperature of the concrete must be higher than for mass construction. Experience shows that for the former a delivery temperature of 70° to 80° F. is satisfactory, while for the latter a temperature of 45° to 50° F. may be used safely, and a temperature somewhere between these two extremes, 45° and 80° F., will take care of practically every kind of job in the coldest weather in which it is practicable to place concrete.

There is nothing to be gained and much to be lost by heating the concrete to a temperature higher than is absolutely necessary. In the first place, it is uneconomical, but more important, it has disadvantages which have a practical bearing on the quality of the concrete produced. In any enclosed space in which concrete is being placed, the hot concrete will tend to produce a fog that will interfere greatly with its efficient and proper handling and this is especially true where the space in which the concrete is being placed is small. The warmer the concrete, the faster it loses its plasticity and the higher are the temperatures which it reaches during its early hardening. These high temperatures greatly increase the amount of water required to cure the concrete, an objectionable feature in winter work where any escaping water means ice, and ice means an increased accident hazard and extra labor for its removal. When the concrete cools off these higher temperatures also cause excessive shrinkage, which makes it difficult to get tight joints with adjacent sections and probably leaves the concrete with internal strains.

HEAT LOSSES IN TRANSPORTATION

The usual means of transportation from the mixer to the job are by chutes or wheeled containers such as buggies, dump cars, etc., although belts are sometimes used. Most of this equipment is of metal, transmitting heat readily, and it would seem reasonable to suppose that concrete so handled would have a considerable drop in temperature during its trip from the mixer to the forms. Tests made by the authors on a number of different jobs do not support this view, for in many cases the drop is so small, even in sub-zero weather, as to be practically negligible. The explanation seems to be that on most jobs the mixing plant is so close to the job and the time elapsing between mixing and placing is so short that the concrete has little opportunity to lose its heat.

In handling concrete through chutes, there are bound to be some heat losses due to the large area of surface exposed per unit of volume.

In actual practice, however, these losses are not great due to the necessities of the equipment. In winter, even more than in the summer, the successful operation of chutes depends on maintaining an almost continuous flow of concrete. In practice, this is seldom obtained and the result is that with temperatures approaching zero it is necessary to protect the chutes and heat them to prevent concrete freezing to them and blocking its flow. Thus, in the usual winter operation, exposed chutes must either be at a minimum, and limited to short lengths, or the chutes must be covered and heated. In either case, the heat losses are negligible.

Handling concrete by means of belt conveyors is also subject to inherent difficulties in cold weather which in most cases will necessitate heating. The principal difficulty is caused by a small film of moisture and cement that is left on the surface of the belt after the concrete has been dumped, which readily freezes if given an opportunity. Scraping the belt at the point of discharge will remove the concrete, but not the film. With short belts, or in mild weather, this film has no chance to freeze because the belt is kept warm by the heated concrete, but in cold weather long belts have to be enclosed, at least for the greater portion of their length, if they are to operate satisfactorily.

In recent years the use of ready-mixed concrete has added a third party, to the group of those responsible for the quality of the concrete used. In cold weather the contractor, as purchaser, orders from a ready-mixed concrete plant concrete of a certain temperature as well as quality. The producer's problem is to deliver the concrete to the job at the required temperature, and frequently the job may be several miles away from the producer's plant. Invariably, the concrete is transported from plant to job in metal containers and in some cases it is mixed in transit by revolving drums. It would seem that the concrete here has an excellent opportunity to lose much heat. Practically, however, the losses are not great. Ready-mixed concrete is usually transported in units of two, three, four or more cubic yards and the elapsed time in transit seldom exceeds 30 to 45 minutes. In urban work, concrete is seldom placed in extremely cold weather, so that ordinarily the temperature losses between plant and job do not exceed 5° F. For hauls under a half-hour and temperatures above 10° F., practically no allowance need be made for heat losses in transit, and with a 10° F. margin in temperature, concrete can be hauled for an hour in zero weather and still arrive on the job at the temperature specified.

Summing up the discussion to this point, it would seem that it is seldom necessary for the temperature of the concrete at the mixing

plant to exceed 80° to 90° F., while generally the temperature may safely be much less, say 50° F. for mass and 70° F. for reinforced concrete building construction. Let us now consider ways and means of providing these temperatures in the freshly mixed concrete.

TEMPERATURE OF THE CONCRETE MATERIALS

The heat to be supplied will, of course, depend upon the temperature of the ingredients before processing starts—the temperatures of the cement and aggregates as received at the mixing or batching plant.

Cement

It is difficult to give any average temperature for cement as delivered, which would be generally applicable in estimating the heat requirements of concrete, for the temperature of cement varies with the method of shipping, whether in bulk or sacked, the length of time in transit and storage and in the method of storage. Freshly ground cement holds considerable heat and, due to its finely divided state and large volume of entrained air, gives up that heat slowly, particularly when stored in bulk in large quantities. Bulk cement in transit in sub-zero weather for several days has been found to have temperatures of 60 to 120° F. when delivered to the job, and sacked cement, when unloaded from cars, will not be very much cooler. Sacked cement stored in unheated buildings will tend to approach the outdoor temperature except that it will not be subject to the same daily fluctuations and if in any quantity, will retain part of its heat for a long time. It will generally be found that even stored cement, unless kept in small quantities over long periods will have temperatures above 32° F., and that for purposes of estimating the heat to be supplied to the concrete mixture to obtain concrete of a given temperature, it is safe to assume for the cement, except in the greatest extremes of cold and long storage, a temperature of 32° F., while for cement used directly as received from the cement mill or silo, or cement handled in bulk, a temperature of 40° to 70° F. can safely be used.

Aggregates

The temperature of the aggregates arriving at a job may be almost anything, but as delivered to the concrete mixer, they must be free from frozen lumps in order to be handled and measured. To be free from frozen lumps, sand must be thawed as the ordinary sand is delivered with from 3 to 5 per cent of contained moisture and will readily freeze. Gravel containing less moisture will freeze less solidly but still will have to be thawed. Either sand or gravel, when heated to remove frost, will have temperatures in excess of 32° F. and, usually, above 40° F.

Crushed rock is different. If used directly from a crushing plant without storage in stock piles, it will usually have temperatures above freezing due to the heat generated in crushing, which it loses slowly because of its low conductivity. If stock-piled after being crushed it does not freeze readily except at the surface since it contains little or no moisture, but if it becomes wet; it behaves like gravel and must then be thawed.

In large unheated stock-piles, the temperature of the interior will depend upon that of the aggregates when the piles were built but near the surface, and in small piles, the temperature will not be much different from the average ambient temperature of the preceding two or three days.

HEATING THE CONCRETE

Basically, the problem of heating concrete is one of supplying sufficient heat units to raise the materials from which it is made from their initial temperature as received at the concreting plant to that required of the concrete when first mixed. Of the different ingredients used in concrete, water is the most readily heated and, bulk for bulk, will store more heat than the other ingredients; hence it offers the easiest means of introducing heat into the mixture. For example, a cubic foot of water will store 62.5 B.T.U. for each degree Fahrenheit rise in temperature, while the average aggregate, whether dry or damp, will store about 25 B.T.U. or less than half as much and in addition this heat will be much more difficult to introduce into the aggregate than into the water.

Obviously it is an advantage, both practically and economically, to use water as the medium for supplying any heat deficient in the concrete, from which follows a basic principle of cold weather concreting: all possible heating of the concrete should be done by means of the water added to the mix and only when the heat available in the water is not enough, should more heat be supplied to the aggregates than is necessary to maintain them in a usable condition.

It is surprising how seldom it is necessary to add heat to a concrete mixture beyond that which can be obtained from the water, in fact with the ordinary mixtures used and temperatures encountered, water can be made to furnish all the heat necessary for concrete temperatures up to 65°-70° F.

Take as an example concrete containing the following weights of materials per cubic yard: cement, 525 lb; fine aggregate (dry rodded), 1280 lb; coarse aggregate (dry rodded), 1930 lb.; water (total) 310 lb. Assume that the fine aggregate has a moisture content of 5.0 per cent,

the coarse aggregate, of 0.5 per cent—both by weight; that the temperatures of the cement and the fine and coarse aggregate at the mixing plant are 60° F., 40° F. and 32° F. respectively. Let us use for the specific heat of both the cement and aggregates, 0.22, the value proposed by Irwin.[1] Assume the temperature specified for the concrete to be 65° F. Under these conditions, the amount of heat required to raise the cement, the aggregates and the moisture contained by the aggregates from their initial temperature to 65° F., would be 23,650 B.T.U. To supply this heat by the water alone requires that the water be at 165° F.

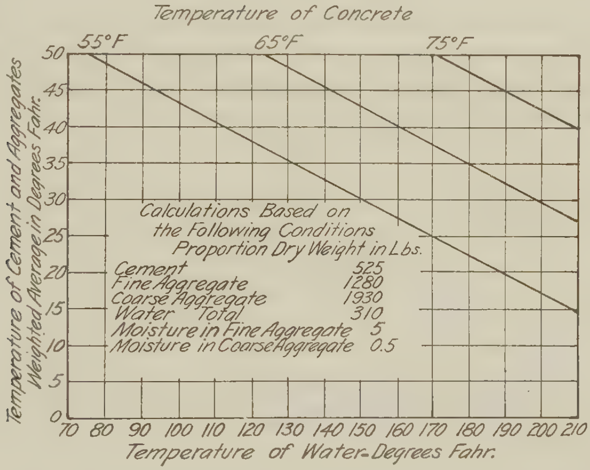

FIG. 1—TYPE OF CHART USED FOR DETERMINING TEMPERATURE OF WATER REQUIRED TO PRODUCE CONCRETE OF A DESIRED TEMPERATURE WHEN AVERAGE TEMPERATURE OF CEMENT AND AGGREGATES ARE KNOWN

For Example—if concrete is wanted at 65° F.and the average temperature of the material is 40° F., the water will have to be heated to 160° F. if the materials are under the conditions noted on the chart.

A simple way of estimating the approximate necessary temperature of the water to heat concrete to different degrees when the average temperature of the aggregates is known, is by means of a chart similar to Fig. 1, which was calculated for the same mixture used in the previous example. Such a chart is only approximate, but as the cement has practically the same specific heat as the aggregate, it is not greatly affected by changes in the proportions, the principal source of error being the difference in the moisture content of the aggregates. At best the maximum error in the temperature of the water is not likely

[1] *Concrete*, Vol. 38, No. 1, p. 24, January, 1931.

to exceed 5° to 10° F., which is close enough for the purpose for which such a chart is usually employed. It is introduced here principally because of the clear way in which it shows the truth of the authors' earlier statement that for most of the conditions met with in practical work, sufficient heat to raise the temperature of the concrete to the required point can be supplied by water alone.

METHODS OF HEATING THE CONCRETE MATERIALS

This paper would hardly be complete without some reference to methods for heating the concrete materials. However, the authors intend to present no detailed directions for doing this, but rather to outline the principles involved, for while the principles are simple, there are usually several ways of applying them correctly, and in any given case the best is usually the cheapest way that under the particular circumstances will give the required result satisfactorily.

Water

Water is the simplest to heat of the materials used in concrete and ordinarily there is no difficulty in providing a satisfactory source of supply. Water may be heated economically in a variety of ways. A small boiler may be installed close to or in the mixing plant, and hot water used directly from it. This is a very satisfactory arrangement on small operations. Where the boiler supplies steam for other purposes, it is usually more satisfactory to heat the mixing water with steam rather than to take hot water directly from the boiler. This can be done in a number of ways, as for instance, steam coils may be used to heat water in a tank or live steam may be jetted directly into it. It is not difficult to devise a satisfactory arrangement but there are certain pitfalls to avoid, of which one of the most common is to use too small a tank, with the result that it is impossible to maintain an adequate supply of sufficiently hot water.

A very satisfactory arrangement for ready-mix or other large plants is a thermostatically controlled hot water supply similar to that used in hotels and apartment houses. The temperature of the water can be exactly maintained and varied at will as required by operating conditions. Such equipment is automatic in operation, costs little to maintain, and is economical in steam consumption.

Whatever the method of heating the water, the important thing is an adequate supply at all times from a plant that can maintain this supply at the desired temperature during the coldest weather with the mixing plant running at full capacity. Unlike other units in the mixing plant, it is seldom feasible or economical to maintain a large storage of hot water and the capacity of the water-heating system must be based

on the peak demand over one or two hours. For example, a one-cubic yard mixer running 40 batches per hour and using concrete materials under the conditions of Fig. 1, would require the heating of 9,400 lb. or more than 1100 U. S. gal. of water per hour. A mixing plant such as is used on many large power developments, equipped with a pair of 2 cu. yd. mixers might need as much as 23,500 lb. or nearly 2800 U. S. gal. per hour and a busy ready-mix concrete plant may have to handle even more than this. A 1000-gal. hot water tank is a large storage, and a 5000-gal. tank is possible only on the largest operations, yet the latter would give less than a two-hour supply for a 100-cu. yd. per hour plant—not a very large reserve.

Heating Aggregates

Aggregates present a heating problem entirely different from water for their specific heat is low and they cannot be heated quickly in any quantity. Further, the volumes to be heated on even a moderate-sized concreting operation are considerable, for instance—a one-cu. yd. mixer turning out 250 cu. yd. per day—a not uncommon condition—would use roughly 170 tons of sand and 250 tons of stone or gravel, a total of 420 tons of aggregate, and some of the larger winter operations reported to this Institute[2] used as much as 3000 tons of aggregate each 24 hours.

Fortunately, as already shown, it is seldom necessary to heat the aggregates more than enough to free them from frost, but even this job should be carefully planned if it is to be done expeditiously and economically.

All methods of heating aggregates divide themselves into two classes —those that use dry heat and those that use live steam. Both systems have their uses, but methods employing dry heat are the least efficient of the two because the transfer of heat from particle to particle must be made either through one or the other of two materials, the aggregate or the air occupying its interparticle spaces, both of which are of low thermal conductivity. Using live steam, advantage is taken of these interparticle spaces to surround each piece of aggregate with a film of hot vapor or moisture which readily loses its heat to the enveloped particle, while the condensed steam trickling back through the aggregate gives up the last available B.T.U. and in addition helps to keep it moist.

Dry heat, besides being slow, has certain other disadvantages. It heats very unevenly; the aggregate adjacent to the points of applica-

[2]"Concreting Methods at Chute a Caron Dam," I. E. Burks, JOURNAL, Amer. Concrete Inst., Feb. 1930, *Proceedings* Vol. 26, p. 315. "Concreting Problems—Chats Falls Power Development," H. L. Trotter and Wilfrid Schnarr, JOURNAL Amer. Concrete Inst., Feb. 1933, *Proceedings* Vol. 29, p. 249.

tion will be very hot and dry while a few feet away, the aggregate will be hardly warm. Aggregates heated in this way will often vary so much in moisture content within a few batches that it is almost impossible to control properly the consistency of the concrete, and aggregate that is very hot and dry will generate steam when it meets the hot mixing water, causing an undesirable fog in the vicinity of the mixer.

The principal disadvantage claimed for heating aggregates by means of live steam is in the supposed accumulation of the condensation, said by some to be variable in amount and hard to control. Experience with many concrete plants in which aggregates have been heated with live steam does not bear out this contention. Except for a bottom layer, where drainage accumulates, a steamed pile will not vary greatly in moisture content from place to place and aggregate reclaimed from it, either from the top by clamming or from the below by tunnels, will be found to be almost as uniform in moisture content as similar piles operated under summer conditions.

A point in connection with the use of live steam is that it is better to use steam under considerable pressure, say from 75 to 125 p.s.i. than low pressures of 5 to 10 lb. With a gauge pressure of 100 p.s.i. steam is emitted at a temperature of approximately 335° F.; with a pressure of 10 lb. at a temperature of about 240° F. and the extra heat available amounts to about 1500 B.T.U. per Imperial gallon of water introduced into the aggregate. At the higher temperature the steam will penetrate farther, do its work quicker and the volume of water that has to drain from the aggregate will be less.

THAWING AGGREGATE IN CARS

Crushed rock or gravel has seldom to be thawed to be handled, but sand, containing as it does 3 to 5 per cent moisture and up, freezes readily if the temperature drops substantially below 32 deg. F. Different methods have been used for thawing sand in cars or barges. Steam points may be driven into the sand and live steam jetted into the mass until the surrounding material is thawed. These are effective but limited in their application. The plan followed at Chats Falls and reported to this Institute by Trotter and Schnarr* was very effective. Standard bottom dump hopper cars were fitted with perforated steam pipes. When these cars were received at the job they were covered with tarpaulins and the pipes connected with a steam supply. In this way a 50-ton car could be thawed and heated in from six to ten hours.

* —Loc. cit.

HEATING IN STOCK PILES

Stock piles also may be heated by steam points driven in wherever it is desired to reclaim the stored material, but for large or permanent plants it will be found handier and more economical to provide a system of steam pipes beneath the piles. The arrangement and number of these pipes will depend on the method followed in reclaiming the aggregate, whether from below or from the surface.

From a heating standpoint, the most efficient system of reclaiming the aggregate is by means of a belt conveyor operating in a tunnel running axially along the length of a slightly elongated stockpile. Where this system is followed, there is no necessity to keep the whole pile thawed, but only that portion immediately above and on either side of the reclaiming tunnel. A line of steam pipes parallel to each edge of the tunnel and another, two or three feet back, with other pipes around the gate openings to the belt will be sufficient. The top of the reclaiming tunnel should extend 6 in. to a foot above the surface of the ground to provide drainage from around the gates and the top and sides should be waterproofed. The top of the pile may freeze but as material is drawn off from below, it caves in and the frozen lumps are thawed out before they reach the gate. It is essential in this type of storage to keep the pile well trimmed and not permit large cavities to form over the gates, particularly with gravel or stone, for if there is too little aggregate over the steam pipes, steam bleeds through, wasting heat and drying the aggregate in spots so that trouble ensues in controlling the water content of the concrete made from it.

Where the layout requires that the aggregate be reclaimed from the top of the pile, the most satisfactory arrangement is to bury parallel lines of steam pipe, spaced on about 4-ft. centers and slotted or drilled every 12 or 18 in. For permanent installations these pipes may be advantageously laid in slots in a concrete slab. If a big stockpile has to be kept warm, it is advisable to divide the system of pipes into sections controlled by separate valves so that only those parts of the pile in use need be kept under steam.

It is also advisable, where possible, to cover stockpiles that are being heated with tarpaulins to conserve the heat, especially where they are shallow and the aggregate is being reclaimed from the surface.

When material has to be reclaimed from the exposed surface of a stockpile, the whole pile has to be kept warm enough to prevent serious freezing. The average temperature of such a pile will therefore need to be considerably higher than that of a pile of similar size which is reclaimed from below; also the heat lost by radiation will be some-

what greater so that more heat will be required by the former than the latter for the same results.

Aggregates to be stored for a time in bins before going to the mixer need not be entirely free from frozen lumps when reclaimed as, unless their passage through the bins is only a matter of minutes, the lumps will be thawed out before they reach the bin gates. In fact, even if a few lumps reach the mixer, no other harm would result than a drop in the temperature of the concrete, for the lumps would be thawed before the mixing operation was over. However, it should be remembered that it takes 144 B.T.U. to melt a pound of ice, that is—it takes practically all the extra heat that can be stored in one pound of water to melt one pound of ice so that a very few pounds of ice in a mix will seriously lower the temperature of the concrete. Also, frozen lumps have a way of getting caught in bin gates.

HEATING AGGREGATE IN BINS

Three conditions govern the amount of heat that must be supplied to aggregates stored in bins. The first is that the principal purpose of a bin is to provide an intermediate storage between the stockpile and the mixer. The second is that the aggregates delivered to these bins must be sufficiently free from frost to handle, and the third is the hazard and inconvenience caused by aggregates freezing while stored in a bin. Generally in a bin, it is necessary only to provide a heating system that will prevent the aggregates from cooling while passing through or remaining in the bin. The most convenient method of heating for a bin is, of course, a system of steam pipes spaced around the sides and bottom, and the steam may or may not be jetted into the aggregate. This is one effective use of dry heat, and the experience of the authors has been that the use of jetted steam in bins is generally undesirable and unsatisfactory because of the almost inevitable trouble caused by the condensation, which collects in the bottom of the bins and causes wide variations in the moisture content of the aggregates so that control of the quality of the concrete becomes very difficult, or impossible. There is also frequent trouble from dripping condensation to the working spaces below, interfering with the workman, damaging the cement and equipment, and in causing accidents due to ice accumulations on floors and stairs.

BOILER CAPACITY

The amount of heat that has to be available to operate a concrete plant during cold weather is a variable that depends on the type of operation, the minimum temperature expected, the peak demand on the plant and the method of storing the aggregates. Where the

aggregates can be reclaimed from below an installed boiler capacity of from 1¼ to 1½ B.h.p. for each cu. yd. of maximum hourly demand will be ample for all operating conditions down to zero or below; where the stockpiles are reclaimed from the top up to 2½ B.h.p. per cu. yd. may be required, depending on the size of the plant, the arrangement of the stockpile and the necessity for heating either one or both aggregates. These estimates are based on the use of the boiler for heating the concrete and concrete materials only, and not for the protection of the concrete after being deposited. It is a mistake to install too small a boiler, but it will be found that these figures will provide a sufficient margin for the usual exigencies of cold weather operation.

For such discussion of this paper as may develop, readers are referred to the JOURNAL *for October, 1934 (Proceedings, Vol. 31). Discussions should be available to the Secretary by August 1, 1934.*

This discussion will be combined with that of the companion paper, by the same authors, which follows.

Cold Weather Protection of Concrete*

BY R. B. YOUNG† AND WILFRID SCHNARR†
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The cold weather protection of concrete is essentially a problem of providing it with proper curing conditions. Curing, like other chemical processes, proceeds less rapidly as the temperature decreases so that in cold weather it will require heat as well as moisture. Today the proper curing of concrete is considered by well-informed engineers as one of the most important factors in making it durable and the purpose of this paper is to stimulate interest in the subject, in the hope that others will be induced to add their observations to the far too meagre and scattered information that now exists.

Protection of concrete in even the most severe weather is not difficult if its principles are understood and practiced. These principles are few but important. One is to remember that the weather is capricious and it is fundamental to successful cold weather concreting that one always be in readiness to provide protection immediately when needed, which means having it ready in advance, and also having it adequate to the temperatures that may occur. Too often, protection is left carelessly until concreting is finished,—usually late in the day with the thermometer dropping—resulting frequently in concrete that is partially frozen before proper protection is given.

It is a good practice to consult the nearest weather bureau regarding the monthly minimum temperatures in the locality of the work. The authors have done this with important work and found the information an excellent guide in deciding how far to go in providing advance protection at any given time.

The protection of any piece of concrete will vary with its size and purpose. Reinforced concrete generally will require much more protection than mass construction such as walls, dams, etc., concrete that is to be highly stressed more than concrete that is not, and concrete

*2nd of two papers by these authors on cold weather concreting presented at the 30th Annual Convention, American Concrete Institute, Toronto, Feb. 20-22, 1934.

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that is to be exposed to weathering more than concrete that is to be totally enclosed. For example, in the ordinary gravity type retaining wall, weight not strength is the principal requirement of the design and the stability of such a wall is ensured once the concrete has hardened, hence it is not necessary in this case to provide artificial curing for the purpose of obtaining in a short time concrete having a high percentage of its design strength, but the top and exposed surfaces of that wall have immediately to resist alternate wetting and freezing and must be protected until they have the quality necessary to withstand this action. On the other hand, the reinforced concrete frame of a building must be protected and cured until it has acquired sufficient strength to carry the loads immediately expected.

HYDRATING CEMENT GENERATES HEAT

Measurements at the center of several large masses of concrete placed at 70° F. showed maximum temperatures of about 160° F. about 72 hours after placing. Assuming that in the center of this large mass the concrete had lost practically no heat, and this assumption is not greatly in error, then the heat generated by the cement must be approximately 180 B.T.U. per lb. Laboratory tests made by the manufacturer gave 190 B.T.U. per lb. for the same cement, a very satisfactory agreement, so that an average concrete containing five sacks, or approximately 450 lb. of this cement per cubic yard would generate in the first 72 hours some 82,300 B.T.U. of heat—a very considerable amount. These figures are probably typical of the majority of our present-day standard portland cements.

It would seem that this tremendous store of heat could be utilized for curing. It can, but unfortunately it is not immediately available. The hardening of concrete is a gradual process, and at the start the heat available at the surfaces of even large masses is not sufficient to counteract the cold and for the first 24 hours after placing cannot safely be depended upon to supply heat for curing.

MASS VERSUS REINFORCED CONCRETE

In this paper, concrete will be divided arbitrarily into two classes, mass and light reinforced concrete, the latter assumed to have sections less than two feet thick, the former, heavier sections.

A fundamental difference between mass and reinforced concrete frame construction is that in the former, the surfaces of the concrete are usually exposed to weathering, while in the latter they seldom are. Another fundamental difference is that the only quality immediately

necessary in the average mass concrete is a resistance to frost action, while reinforced concrete has to be cured for early strength.

In mass construction, the heat generated by the setting cement may be utilized, under certain conditions, after the temperature of the concrete begins to build up, but in small structures, the surface being proportionately much greater with respect to the mass of the concrete, the quantity of heat available is not sufficient to be an important factor in curing.

These factors affect the methods of curing applicable in the two cases and alter the approach to the problem.

The protection and curing of reinforced concrete construction is well understood and as it has already been described by Johnson[1] this paper will discuss more particularly the protection of mass concrete and the general principles of fitting protection to the exposure, with only incidental references to the methods usually followed in curing structural frames.

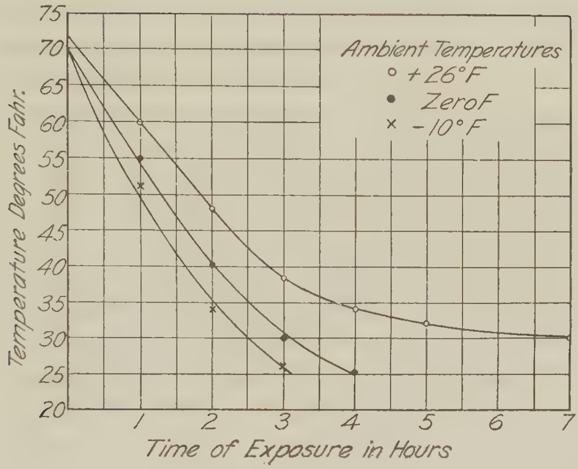
PROTECTION REQUIRED

Definite data are lacking on the actual amount of protection necessary to safeguard concrete under different conditions. The authors have tried to meet this lack by experiment and observations in the field, but the information they have obtained is, at best, sketchy. However, incomplete as it is, it offers some quantitative data that may be useful to others.

Obviously, the amount of protection will vary with the ambient temperature. This is very clearly indicated by the experiments of Fig. 1. Concrete protected by $\frac{7}{8}$-in. wooden forms was subjected to different temperatures. In each case, the initial temperature of the concrete was 70° F., and its mass was so small that the heat generated in setting was negligible. Measurements were taken with thermocouples placed on the surface and at a depth of approximately $1\frac{1}{4}$ in. The tests indicated that at an ambient temperature of 26° F. concrete placed at 70° F. will be prevented from freezing by $\frac{7}{8}$-in. forms for six hours; that at 0° F., this period will be reduced by half; and at —10° F., the forms alone would still protect the concrete from actual freezing for two hours.

However, as has been said, the curing of concrete in cold weather is more than merely preventing it from freezing, and in most cases some artificial heat must be supplied to maintain it at a proper temperature.

[1]"Winter Concreting Methods", Report of Committee 604, Robert C. Johnson, Author-Chairman, JOURNAL Amer. Concrete Inst., Feb. 1930, *Proceedings* Vol. 26, p. 397.



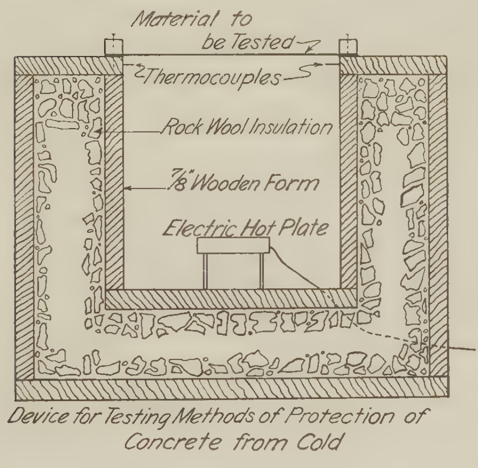


FIG. 1—RATE AT WHICH CONCRETE COOLS

FIG. 2—DEVICE FOR TESTING METHODS OF PROTECTION OF CONCRETE FROM COLD

Most specifications require that concrete be cured at a temperature of at least 50° F. for not less than 72 hours. Experiments by the authors have shown that less than this amount of curing will not produce concrete resistant to frost action and that for concrete which is to be highly stressed in service, 72 hours may not be sufficient to develop by a proper margin the strength necessary for carrying the imposed loads.

The degree of protection required is practically independent of the class of concrete used—concrete whether rich or lean, dry or wet, if placed at the same temperature will contain practically the same quantity of heat units. While both the rich and the dry concretes will generate appreciable heat sooner than the lean or wet, the differences, while measurable, are neither great enough nor dependable enough to take into consideration in planning protection.

In cold weather, good practice requires that concrete be deposited in the forms at a temperature that will prevent it from freezing until other protection can be provided. The initial temperature of the concrete will not appreciably affect the amount of protection that will ultimately be required, but it will change the interval that may safely elapse between the placing of concrete and its protection. For this reason, the authors recommend minimum temperatures of 60° to 70° F. for all concrete placed in light sections as these cool rapidly because of the relatively high ratio of exposed surface to volume, and since in mass construction this relationship is reversed, concrete placed in large concentrated volumes may safely be deposited at 50° F.

In protecting concrete in cold weather, one is attempting to prevent, first, the loss of heat in the concrete, and second, the penetration of cold from outside. Protection, therefore, is largely a problem of heat insulation and is subject to the same general principles. Different degrees of protection will be required at different ambient temperatures to safeguard the concrete and conserve heat and the methods by which this can be done are similar in many respects to those used to solve other heat insulation problems.

Steel forms, while they may have many advantages where repeated re-use is possible, are at a disadvantage in cold weather because they are such excellent conductors of heat. Being braced or supported by steel members they have a further disadvantage that there are usually many parts projecting into the outside air which dissipate heat. Therefore, steel forms, unless their other advantages are very great, should be avoided in work subject to extremely low temperatures.

Wooden forms are more generally used for concrete construction than steel, and are preferable from the standpoint of protecting concrete from cold as wood has a substantial insulating value. This insulating value depends on the moisture contained in the wood and consequently dry forms give better protection than those that have become saturated with water. As would be expected, increasing the thickness of the form sheeting adds to its insulating value, but it is

doubtful if the extra cost of the heavier forms can be justified from the standpoint of protection alone, as equal or better insulation can be provided in other ways at less cost.

A very effective thermal insulator is a dead air space. It does not have to be wide, in fact a space ¾ to 1 in. thick is probably more efficient than one that is wider because the narrower space permits less circulation of the entrapped air. Experiments show that a dead air space between two ⅞-in. boards will protect mass concrete placed at 70° F. down to a temperature of about 10° F., but to attain this result the air in the space between the boards must not be allowed to circulate. A somewhat poorer result can be had by tacking tarpaulins or other covering to the outside of the forms, but here it is difficult to provide a truly dead air space because the distance that must exist between sheeting and any covering placed over the walers, makes some circulation of air almost a certainty. Further, it is very difficult to fasten tarpaulins so tightly to the forms that there will not be drafts and the authors' experiments show that if openings exist in the outer covering to as little as 1½ per cent of its area, the efficiency of the air space as a means of protecting the concrete has been reduced by more than 15 per cent.

Not only does the presence of openings into the air space reduce the protection, but the practice of permitting reinforcing rods or other steel to project through the forms and coverings into the cold air is very wasteful of heat. It will also be found to cause the concrete surrounding the steel immediately back of the forms to freeze when sub-zero temperatures exist. The experiments described later, indicate that if as little as 1⅔ per cent of an area is taken up by projecting steel, it will cause an increase in the heat losses over this area of approximately 35 per cent.

EFFICIENCY OF PROTECTIVE METHODS

To determine the efficiency of different methods used for supplying protection to concrete surfaces an insulated box was made as shown in Fig. 2. The air inside the box was brought to a constant temperature by means of an electric hot plate, the input to which could be measured. The box was well insulated with rock wool so that practically all the heat lost had to pass through the test area at the top. This test area was covered by different materials used in the cold weather protection of concrete and the heat losses they permitted were measured. The results were not absolute because of the design of the top, but they were comparative and indicate the relative value of the different materials. Some of the results are shown in Table 1.

TABLE 1—COMPARATIVE INSULATING VALUE OF VARIOUS MATERIALS USED TO PROTECT CONCRETE SURFACES IN COLD WEATHER

Materials	Relative Insulating Value of Different Materials Used in Protecting Concrete
2 Layers of ⅞-in wooden sheeting with a ⅞-in. air space between	145
2 Layers of ⅞-in. wooden sheeting with a draft through the air space	125
Pine sheeting, 1¾ in. thick	117
Pine sheeting, ⅞ in. thick	100
⅞-In. Sheeting with 1.67 per cent of area occupied by projecting steel	77
Canvas	64
Tar Paper	62
Drier Felt	62
Sisalkraft	61
Building Paper	61
Straw, loose, 4 in. thick	30

These tests show that if the insulating value of ⅞-in. boards is taken as 100, the use of 1¾ in. sheeting would add only about 17 percent to the insulating value of the forms. Splitting this 1¾ inches into two ⅞-in. boards with an air space between would increase the insulating value to 145, but let the air circulate in this air space and it is lowered to 125. A canvas tarpaulin laid directly on the concrete would have a relative insulating value of 64, although if supported so as to give an air space between the concrete and the tarpaulin, the value would probably be about 90, still less than a ⅞-in. wood form. The several types of thin coverings, canvas, sisalkraft, building paper, etc. all have about the same value as protective mediums and the often specified straw covering, when used alone, has a very low protective value, probably because like a wool sweater, the air passes through readily.

Experience has shown that mass concrete placed at 70° F. with only the top exposed, that is, partially protected by surrounding earth or other concrete will be safely protected down to a temperature of about 15° F. by a tarpaulin placed over the exposed surface in such a way as to provide an air space.

For mass concrete, ordinary ⅞-in. forms will protect the concrete from actual freezing down to 20° F. at all but the corners and edges, but these will require additional protection. If a dead air space is provided either by double sheeting the forms or hanging a tarpaulin over them, the concrete will be protected from freezing down to probably 10° F. For lower temperatures, heat must be supplied to the surface, to prevent actual freezing of the concrete while it is setting.

PROTECTING CORNERS AND EDGES

The concrete at corners and edges loses heat much more rapidly than does that at the surface of large areas. This is not surprising

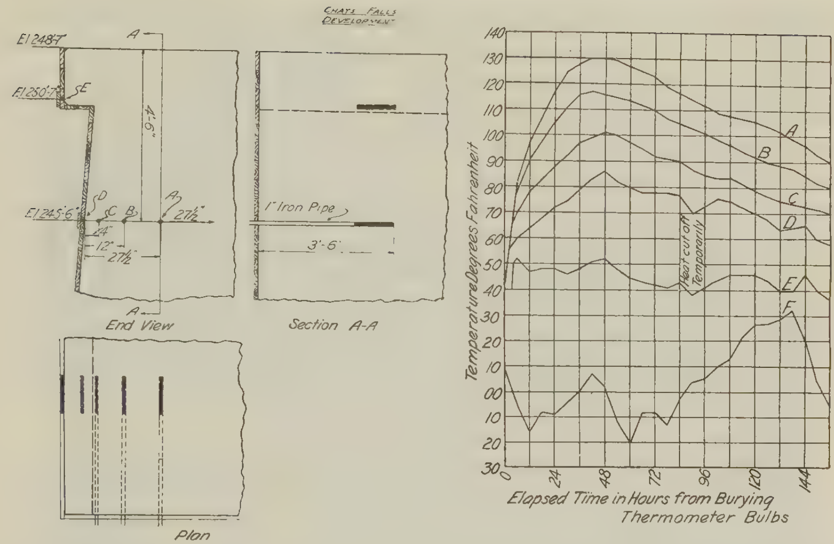


FIG. 3—EXPERIMENT SHOWING THAT EXTRA PROTECTION IS REQUIRED AT CORNERS AND SURFACES

Note: Heat generated in concrete is of little value for protection in first 24 hrs.
(Left) Location of temperature recorder bulbs.
(Right) Temperatures during curing of concrete.

A. 27 in. from surface
B. 12 in. from surface
C. 4 in. from surface
D. At surface under form
E. At surface under forms at corner
F. Outside temperature.

when one considers the greatly increased surface per unit of volume at these points through which heat may be dissipated. As every experienced observer knows, it is at the corners and edges of a structure that usually one first finds serious deterioration, and therefore common sense dictates that at such places special precautions should be taken to see that the protection is ample. In many cases, extra protection may be required, but much can be done by bringing tarpaulins up and over corners and edges, and keeping them well away from all projections. It is also very important to have the protecting covering tight at these places for it is common to have the joints between adjacent tarpaulins come at corners and in many cases openings are left through which not only serious losses of heat may occur, but cold may penetrate and chill the concrete immediately adjacent to the openings.

The cooling of concrete at corners is well illustrated in the experiment shown in Fig. 3, which was carried out under job conditions on a

large pier, 4½ ft. wide. All forms were ⅞-in. and were covered with drier felt, and the temperature of the concrete when deposited was from 70° to 80° F. Thermometers placed on the center line of the pier recorded a maximum temperature of 130° F. in 48 hours, dropping gradually to 95° F. by the sixth day. Another thermometer placed between the forms and concrete on a large surface rose to 85° F. in 24 hours, dropping off gradually to 68° F. at the end of the sixth day. However, a third thermometer placed at a corner beneath the forms never got above 52° F. throughout the six days of the test in spite of dry heat that was being supplied to the outside of the forms in an effort to maintain this corner above 45° F. The ambient temperature was 10° F. when the concrete was placed and dropped to —20° F. at one time.

ENCLOSURES

On many winter jobs it is common practice to enclose the area in which concrete is being placed by a frame-work covered with tarpaulins or in some cases, building paper, drier felt or similar substance. These enclosures are then heated to give the proper temperature for curing the concrete.

The construction of these enclosures is simple and will not be discussed here other than to point out that their most common fault is the prevalence of openings, which reduces their effectiveness greatly. Certain openings are unavoidable, such as entrances for workmen and materials, but these should be kept at a minimum and provided with covers by which they may easily be kept closed when not in actual use. The word *easily* here needs special emphasis for any covering at an opening that is awkward to handle will not be used by the average workman except when he is under observation and hence is of little practical value.

Care should be taken to avoid unnecessary openings into the enclosure. The most troublesome places in this respect are where adjacent tarpaulins or other coverings overlap, where they join the edges of the forms below and where they have to be draped and fitted around obstructions. The openings themselves may be small, but in the aggregate the cumulative effect is to reduce greatly the efficiency of the protection afforded by the enclosure.

Certain characteristics of these enclosures should be borne in mind. The average temperature inside the enclosure may be quite in order, and still in places the concrete will not be getting proper protection and the lower the outside temperature, the greater the likelihood that

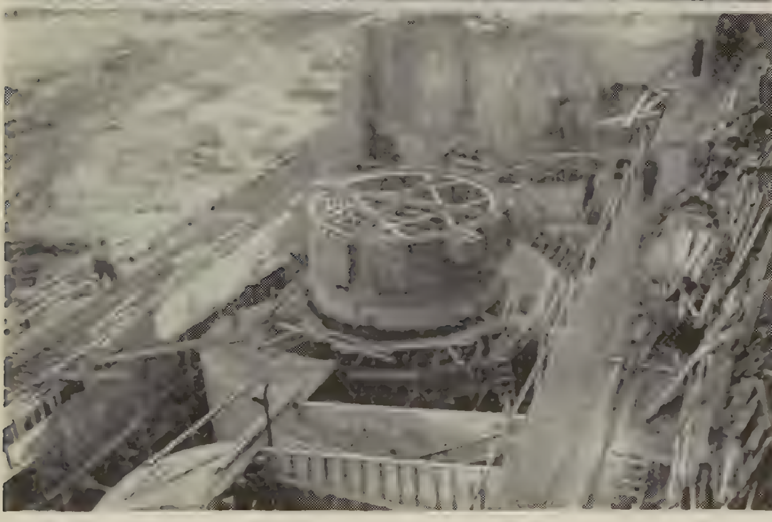


FIG. 4—CONCRETING UNDER LARGE ENCLOSURES
Note: Size of man standing on enclosure.

FIG. 5—TYPE OF INTRICATE FORM WORK UNDER ENCLOSURES IN FIG. 4
Note: Steel projecting through enclosure

this will be true. With the average temperature of the interior at 50° and the outside temperature at 20° F. or higher, a tight enclosure should maintain temperatures above freezing throughout, but at 0° F., there will be a space for several inches inside the walls of the ordinary canvas-covered enclosure that will be below freezing. At −20° F., the outside cold will seem to be flowing into the enclosure

FIG. 6—35-FOOT PIERS CURED WITH LIVE STEAM AFTER CONCRETING WAS COMPLETED

FIG 7—COVERING OVER 600-FT. DECK SLAB—NOTE PROTECTION USED

and all concrete within a couple of feet of its walls will have to be watched to prevent actual damage.

HEATING ENCLOSURES OR FORMS

In most cases, some heat will have to be supplied inside the enclosures. With reinforced concrete frames where the enclosed areas are large and the mass of concrete small, a considerable amount of heat will be required to maintain proper curing conditions. For this type of protection the reader is again referred to Johnson's excellent paper* However, where the mass of concrete that is being cured, is

*Loc. cit.

Fig. 8—Concreting foundation under large enclosure—large entrance which is wasteful of heat

large, the concrete will itself liberate considerable heat and less will have to be supplied.

The common practice in building construction is to use salamanders for heating. These have the advantages of being flexible and independent of any outside sources of heat, such as a boiler plant. They have the disadvantage of tending to dry the concrete, they are gassy, and when used around wooden forms constitute a considerable fire hazard.

On heavy construction, heat is usually supplied by means of live steam, which keeps the concrete moist as well as warm. Its disadvantage is that the same dampness which is good for the concrete is objectionable where men have to work. It is, however, the most satisfactory way of curing vertical faces of walls because it can be so readily introduced between tarpaulin and form. Where men are working, it will be found that a heater equipped with steam coils and a fan by means of which a large volume of hot air can be forced in any direction is very satisfactory. When the workmen are out of the enclosure, the heater may be disconnected, removed and live steam substituted. These heaters are also very useful in directing a large volume of heat into a corner or recess and also to furnish additional heat in those parts of an enclosure most subject to cooling.

In conclusion, the authors point out again that successful protection depends on:

1. Knowing your weather and being prepared for it.
2. Curing reinforced concrete for strength.

3. Curing surfaces exposed to weather for durability.
4. Fitting protection to the weather.
5. Not placing too much dependence on the heat generated by the concrete, at least for the first 24 hours.
6. Watching the places that are likely to be coldest, whether in the concrete or enclosures.

Finally, it is not difficult nor expensive to provide satisfactory curing conditions for concrete in cold weather. Success depends on simple precautions and careful planning, not on elaborate plant, and no one need avoid a winter job merely because of anticipated cold weather.

For such discussion of this paper as may develop, readers are referred to the JOURNAL *for October, 1934 (Proceedings, Vol. 31). Discussions should be available to the Secretary by August. 1, 1934*

This discussion will be combined with that of the companion paper by the same authors, p. 279.

Vibrating Concrete at Pine Canyon Dam*

BY SAMUEL B. MORRIS†
MEMBER AMERICAN CONCRETE INSTITUTE

AND ROSS WHITE‡

When the specifications for Pine Canyon Dam were being written, a clause was inserted requiring the use of vibrators in placing mass concrete. The engineers in charge of the project were convinced that by the use of vibrators they could procure: (1) a more dense concrete; (2) better bond between lifts; (3) better surface finish and appearance; (4) concrete with the required strength and impermeability with less cement, and (5) as a result of (4), a concrete with less total heat rise as a result of the hydration of the cement. It was also thought that these results could be achieved not only without increasing the cost of the work, but at an actual saving, largely through the decreased cement required.

At this writing substantially all of the total concrete required in the dam has been placed, and definite answers can be made to most of the questions raised in regard to the use of vibrators.

A number of samples cut from the mass concrete of the dam after it had hardened, show that the concrete, as placed, has an average weight of more than 156 lb. per cu. ft. This is checked by the weight of hundreds of 14 x 28 in. cylinders made in the regular routine of testing, many of which exceeded 157 lb. per cu. ft. Aggregates from which this concrete is made have a specific gravity slightly under 2.68.

The reservoir has been filled to a depth of 165 to 170 ft. for three weeks and no seepage has yet appeared between lifts. No adequate test of the bond between lifts has yet been made, but all evidence otherwise obtainable points to the existence of a bond decidedly superior to that usually obtained in work of this character.

The evidence as to surface finish is conclusive. On a dam face 325 ft. high and 800 ft. long, not a single honeycomb or "bug-hole" has been found on the upstream face and not over half a dozen such places

*Presented at the 30th Annual Convention American Concrete Institute, Toronto, Feb. 20-22, 1934.
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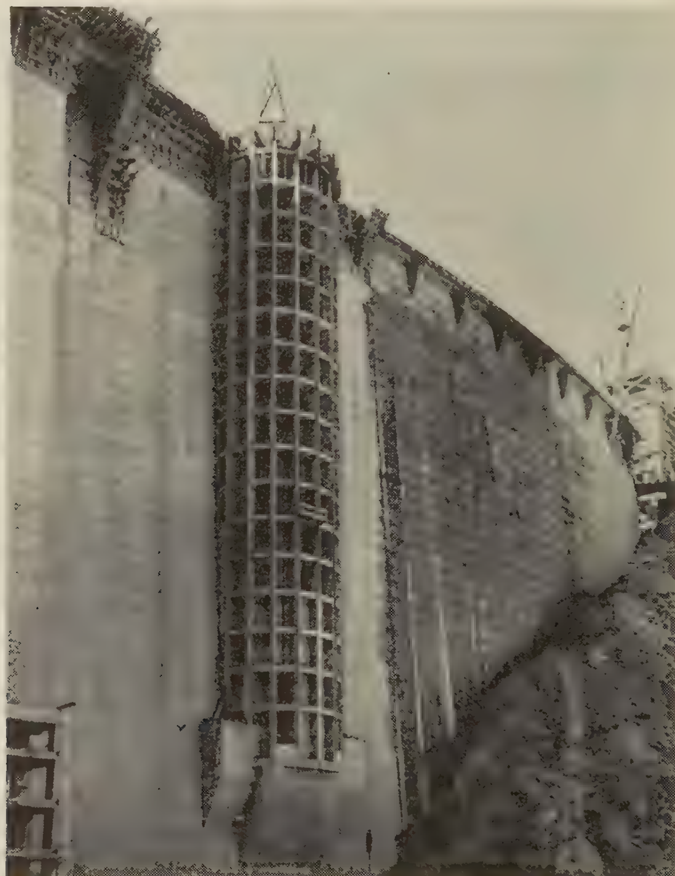


FIG. 1—UPSTREAM FACE OF PINE CANYON DAM, INCLUDING OUTLET TOWER

on the flat (slope .81:1 to .95:1) downstream face. Fig. 1 (upstream face) shows the type of finish obtained. Fig. 4 is a general view of the downstream face of dam, spillway, and contractors' plant. There was a 165-ft. depth of stored water when this picture was taken.

The cement used in Pine Canyon Dam is a low-heat, slow-setting product, produced especially for this project, yet with 0.95 bbl. per cu. yd., strengths of 2,000 p.s.i. at 28 days, and 3,000 p.s.i. at 90 days have been consistently obtained. This corresponds to a strength of 3,000 lb. at 28 days if normal portland cement had been used. These tests are based upon 14 x 28 in. cylinders using full size aggregate

FIG. 2—SPUD TYPE INTERNAL ELECTRICALLY OPERATED VIBRATORS IN MASS CONCRETE WITH SIX INCH MAXIMUM AGGREGATE

FIG. 3—PLATFORM TYPE EXTERNAL ELECTRICALLY OPERATED VIBRATORS IN USE AT PINE CANYON DAM

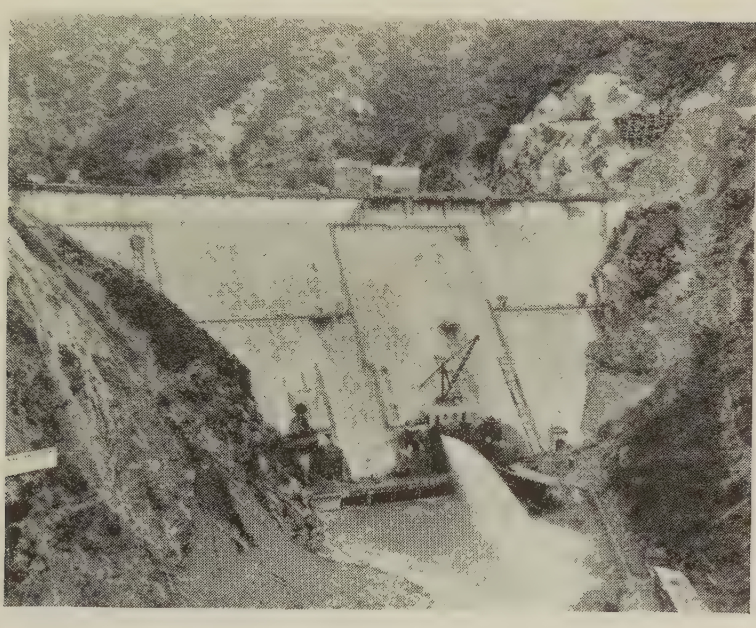
FIG. 4—DOWNSTREAM FACE OF PINE CANYON DAM, SHOWING SPILLWAY AND CONTRACTORS' PLANT. RESERVOIR LEVEL OF 165 FEET BEHIND DAM. (SEE OPPOSITE)

having 6-in. maximum cobbles. Tests on 6 x 12-in. cylinders with aggregate larger than 1½ in., screened out, show strengths 20 to 25 per cent greater than the 14 x 28 in. cylinders.

If sufficient water were used in this mix to make it workable without vibrators, strengths of not to exceed 1600 lb. at 28 days could be expected, together with greatly increased permeability. As a corollary of the above, holding the water cement ratio the same as in vibrated concrete, it would require at least 0.2 bbl. of cement per cu. yd. additional to make the concrete plastic enough for hand placing.

In cement alone it would appear, therefore, that the use of vibrators saved 0.2 x $1.65 x 445,000 = $146,850.

The specified use of vibrators did not increase the cost of placing as evidenced by the bid price of $2.36 per cu. yd., exclusive of cement. This can be compared with $2.50 per yd. for the slightly greater volume in Madden Dam, and $2.70 per yd. for the ten times greater volume in Boulder Dam.



(SEE OTHER HALF OF PANORAMIC VIEW OPPOSITE)

The bid price at Pine Canyon is not unduly low, due to "unbalancing" of bids, as all other items in the contract are correspondingly low.

The average heat rise in the mass concrete at Pine Canyon has been 38° F. at 90 days. If 21 per cent more cement had been used, we could anticipate a corresponding per cent of rise in heat, so it seems justifiable to credit the use of vibrators with such reduction in total heat rise, exclusive of any saving due to the lower water-cement ratio made possible by the use of the vibrators.

Another large saving due to the use of vibrators is the decrease in material lost in cleaning up the surface of each lift. With the low water-cement ratio used, practically no laitance forms, and the platform type vibrators compact the surface to such an extent that only a very thin film of material is lost in the cleanup, as compared to the appreciable fraction of an inch removed when a wetter, uncompacted concrete is used.

Two general types of vibrators were used, one, the spud or internal type, is thrust directly into the mass of concrete (Fig. 2), while the other, the platform or external type, is fastened to a plank about 12 x 48 in., and produces its results by being pressed firmly against

the surface of the concrete (Fig. 3). Both are almost indispensable, but the effectiveness of both is greatly increased by being used in conjunction with each other. With the platform type alone, it is difficult to get 100 per cent efficiency against the forms, especially under the overhanging forms of the down stream face, and there is also a possibility that full compaction is not always secured on unusually thick layers. On the other hand, with the internal type alone, the very violence or effectiveness of the vibration leaves the surface, where there is no weight above to hold the particles down, in a more or less loose condition. Used together, however, each complements the other, resulting in a mass of concrete that is thoroughly vibrated against the forms, and is dense and compact from top to bottom of the lift. Using the internal vibrators first tends to better eliminate internal air. The platform vibrators tend to seal the surface.

At the Pine Canyon Dam commercial power is available at 50 cycles only. This gives 2 cycle motor speeds of 2,900 r.p.m., which was found inadequate for efficient operation of the vibrators. Therefore, a rotary frequency charger was installed to step up the frequency to 75 to 80 cycles, giving motor speeds of 4,400 to 4,700 r.p.m., giving much more efficient results.

The Pine Canyon Dam is being constructed for the Pasadena Water Department, of which Samuel B. Morris is Chief Engineer and General Manager, Ross White (now Superintendent of Construction for the Tennessee Valley Authority on the Norris Dam) has been succeeded by Verne L. Peugh as Construction Engineer of the San Gabriel project. R. W. Spencer is Resident Engineer at Pine Canyon Dam. Bent Brothers, Inc., Winston Bros. Company, and Wm. C. Crowell are the General Contractors for the construction of Pine Canyon Dam.

For such discussion of this paper as may develop, readers are referred to the JOURNAL *for October, 1934 (Proceedings, Vol. 31). Discussions should be available to the Secretary by August 1, 1934.*

Some Tests of Load Capacity of Floors Made with Precast Concrete Joists*

BY R. E. COPELAND† AND P. M. WOODWORTH†

INTRODUCTION

This is a progress report of an investigation of the structural performance of a floor construction consisting of precast reinforced concrete joists with cast-in-place or precast 2 or 2½-in. reinforced concrete slab. This type of construction is not new, such floors having been built 20 years or more ago. Recent interest in its application to residences and other light load buildings has made desirable a more complete understanding of its structural performance under load.[1]

Fig. 1 shows the general construction details of this type of construction as used in a building. The particular joists indicated are of a modified I-beam shape with top and bottom heads to accommodate the main reinforcement and to increase the bonding area in contact with the slab, while the thin web section reduces the weight.

PURPOSE AND SCOPE OF TESTS

The purpose and scope of the tests made thus far are:

1. *Concrete Mixtures:* Investigation of the influence of proportions, consistency, type and grading of aggregate on placeability, strength and appearance of the concrete.

2. *Joint Strength:* Tests on 27 specimens to determine the shearing strength at the joint with bond effected by different means.

3. *Floor Load Tests:* Uniform loading of 12 panels, 14 ft. by 4 ft. and 4 panels 14 ft. by 2 ft. to determine deflection at midspan, strains in concrete and tensile steel, and the general behavior of the panel with load increase.

Variables studied included:

A. Welded joist reinforcements as compared with reinforcements having stirrups hooked around main bars.

*Presented at the 30th Annual Convention, American Concrete Institute, Toronto, Feb. 20-22, 1934.
†Cement Products Bureau, Portland Cement Association, Chicago.
[1]Results of tests on similar types of floor construction are given in: (1) "Shearing Strength of Construction Joints in Stems of Reinforced Concrete T-Beams as Shown by Test," by Lewis J. Johnson and John R. Nichols, *Transactions* A. S. C. E., Dec., 1914, p. 1499; (2) Discussion by F. N. Menefee of "Tests of Plain and Reinforced Haydite Concrete," by F. E. Richart and V. P. Jensen, *Proceedings*, Amer. Soc. for Testing Materials, 1930, p. 700.

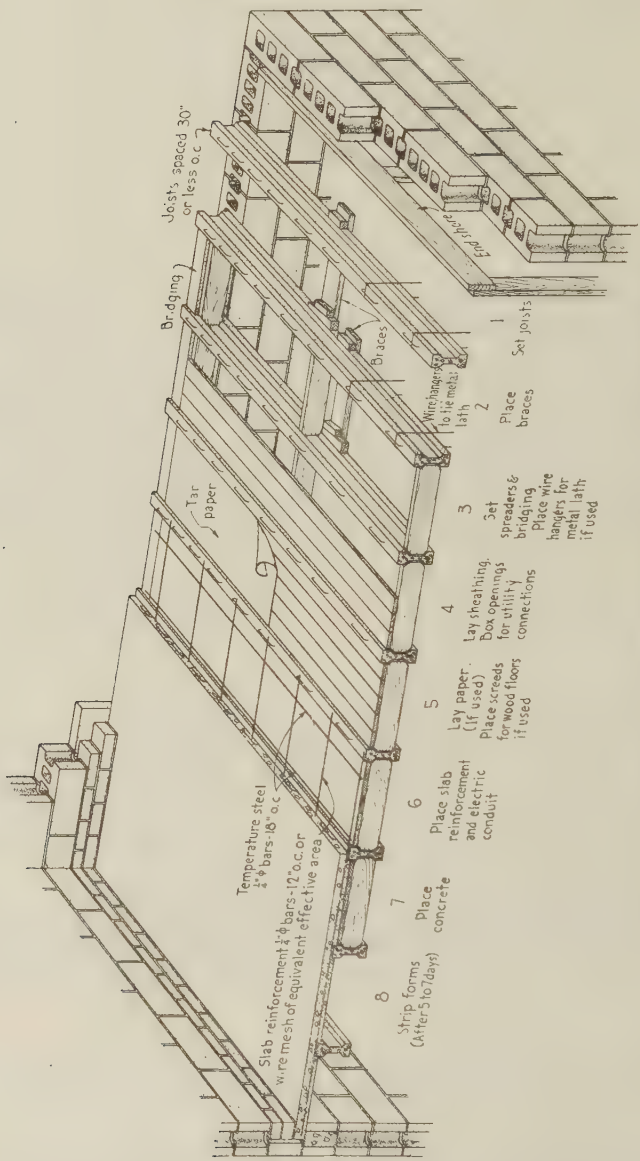


FIG. 1—PRECAST CONCRETE JOIST FLOOR CONSTRUCTION IN PERSPECTIVE

B. Effectiveness of diagonal as compared with vertical stirrups.

C. Type of aggregate used in joist concrete and strength of concrete in joists and slabs.

D. Type of bond of slab to joist.

E. Performance of precast as compared with cast-in-place slab.

CONCRETE MIXTURES

Mixtures ranged from 1:3 to 1:4 by dry rodded volumes. Aggregates included Elgin sand and (No. 4 – ⅜-in.) gravel, fine and coarse Haydite, fine and coarse cinders and combinations of Elgin sand with the Haydite and cinder aggregates. Consistencies ranged from 4- to 8-in. slump. Materials were mixed for two to three minutes in a drum type mixer. For these tests a wood mold was designed to accommodate two 2-ft. lengths of joists at each filling. Approximately 30 of these short joist sections were made. The concrete was placed in three to four layers, each layer being well spaded with a long, pointed trowel.

Observations

1. Due to the thin web section, there was difficulty in placing mixtures in which the maximum size of aggregate exceeded ⅜-in.

2. 1:3 mixtures were substantially more placeable than the 1:4 with all types of aggregates.

3. Cinder and Haydite concrete mixtures were harsh and were improved by substituting Elgin sand for from 10 to 25 per cent of the total volume of the aggregate.

4. To obtain satisfactory placeability with hand spading, it was indicated that the fine aggregate should compose 60 to 75 per cent of the total aggregate volume.

5. Fairly wet consistencies, 6- to 8-in. slump, were essential to rapid placing and satisfactory appearance of concrete.

6. The tendency for shrinkage and settlement to form a hair-line crack at the junction of the web and top head was largely eliminated by more careful compaction of the top layers of concrete and by damp curing.

Subsequent experience with limestone and Waylite aggregates indicates that, in general, these same observations apply to their use.

General Conclusions

This study and later experience in casting 32 joists, 14 ft. 8 in. long demonstrated that satisfactory results can be obtained with hand spading and suitable concrete mixtures. Because of the greater shrinkage however with the wet consistencies, large amount of fine aggregate and richness of mix necessitated by this method of placing,

the use of mechanical means of compacting should receive consideration. Some joist manufacturers are employing vibration with good results in placing considerably stiffer mixtures than used in these tests.

SERIES A—TESTS OF STRENGTH OF JOINT

Strength of bond between joist and slab is an important consideration in the design of precast joist floors. Handability and economy require joists of a light section which ordinarily will not provide an adequate concrete area to maintain compressive stresses within allowable limits. It is essential, therefore, that the joint strength be sufficient to develop satisfactory interaction between joist and slab so that the latter will be available for resisting compression.

Joint strength was investigated by tests on 27 specimens as shown by Fig. 2. There were 6 different types of metal ties, each investigated with and without the addition of concrete bond. Specimens 7 and 7′ had no metal ties. Details of the reinforcements used for each bond condition are shown by Fig. 3.

The joist sections were of Haydite concrete and the slabs of sand and gravel concrete. Details of mixes and concrete strengths are given in Table 1.

Joints with no concrete bond were tested to develop minimum values and also indicate more definitely the relative effectiveness of metal ties. The bonding surfaces of the joist sections of these specimens were heavily coated with a paraffin-rosin solution prior to casting on the slab.

The specimens were capped on the bearing faces, positioned in the testing machine as shown by Fig. 2 and the load was applied through a 6 in. spherical bearing block and 2 in. distributing plate. The downward movement of the joist section was measured at increments of load by 2 Ames dials extending from the loading platform to distributing plate.

Observations and Discussion of Results

The ultimate strength of specimens 1 to 5 depended on the concrete bond. Apparently the metal ties were not sufficiently rigid to become effective at deformations within the limit of the concrete bond. After the concrete bond fractured, the joint strength which remained averaged 75 per cent of the ultimate and undoubtedly was derived from the metal tie as well as frictional resistance.

The metal connection used in specimen 6 gave good results, the ultimate strengths averaging about 75 per cent higher than the load at which the balance beam first dropped suddenly and which was assumed to mark the fracture of the concrete bond.

There was excellent adhesion between the slab and joist concretes. In several tests a thin layer of joist concrete was sheared off with the slab.

Results of the tests are given in Table 2. The total bonding area was 80 sq. in. and total contact length of joint, 20 lin. in. These values were used in reducing ultimate load to joint strength lb. per sq. in. and lb. per lin. in. Results on joints with concrete bond and metal ties ranged from 1,125 to 1,815 lb. per lin. in. Specimen 7, which had no metal tie, developed 1,360 lb. per lin. in. The superiority of specimen 6 perhaps was to have been expected.

The strengths of joints with no concrete bond ranged from a minimum of 175 lb. per lin. in. for specimen 7′ which had no metal tie to a maximum of 1,160 lb. per lin. in. for specimen 6. While the results indicate that only very rigid ties such as type 6 would furnish the requisite joint strength alone, the value of such supplemental connections is more evident than in the first group of tests.

The ratios of joint strength to allowable shearing stress on the web actually indicate safety factors since the horizontal shear would be limited in any case by the allowable

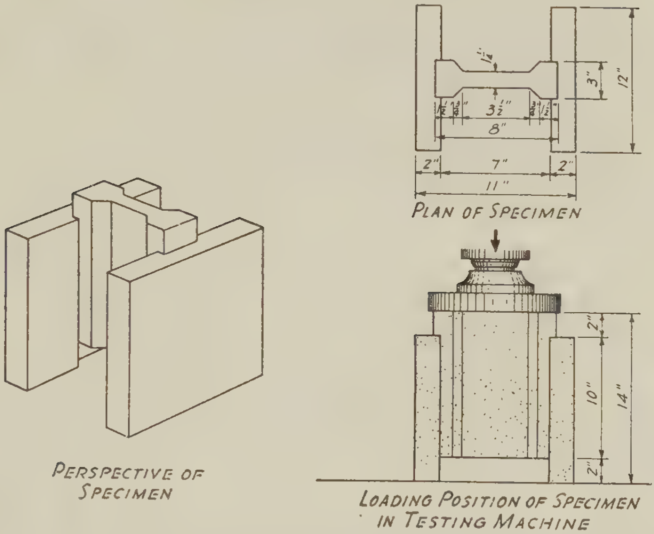


FIG. 2—GENERAL DETAILS OF JOINT STRENGTH TEST SPECIMEN

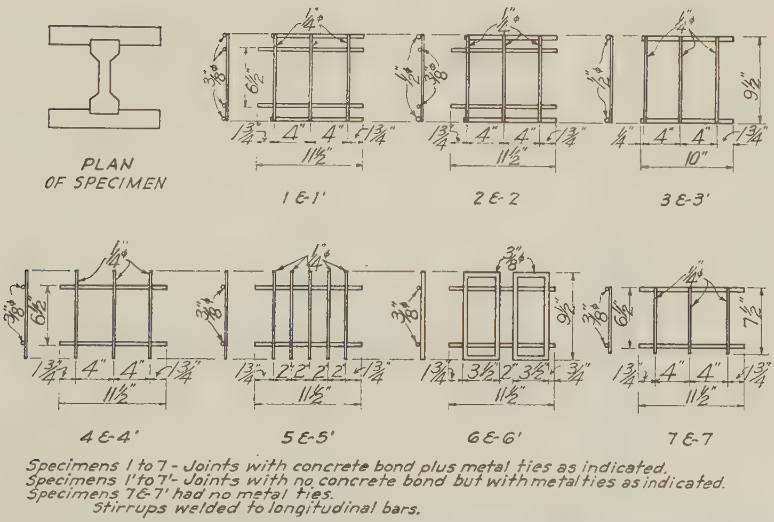


FIG. 3—REINFORCEMENT DETAILS, SPECIMENS FOR JOINT STRENGTH TESTS

TABLE 1—CONCRETE MIXTURES AND COMPRESSIVE STRENGTHS

Compressive tests on 3 x 6 in. cylinders air dry condition at time of test of specimen.
Cylinders cured in same manner as test specimen: 7 days damp, remainder in laboratory air for Series A and B1-1, B2-1; all air curing for rest of Series B.
Age at test: Joist cylinders, 42 days; Slab cylinders, 21-25 days, except that joist and slab cylinders for Panel B11 were 28 days.

Type of Aggregate	Quantities by Dry Rodded Volume Cu. Ft. per Sack of Cement: Separated Aggregate: Sand 0–4	Fine 0–4	Coarse 4–3/8	Coarse 3/8–3/4	Combined Aggregate	Unit Wt. Combined Aggregate lb. per cu. ft.	Fineness Modulus, Combined Aggregate	Compressive Strength p.s.i.	Used in Series
					Joist Concrete				
Haydite	.33	1.75	1.05		3.0	63	4.17	3260(a)	B1-2, B2-2, B2-3, B6, B7, B8, B9, B11, Slab B11.
Cinders	.82	1.58	.75		3.0	81	3.46	2780	B3
Waylite	.81	.81	1.55		2.6	75	4.35	1070	B5
Limestone	1.35(b)	1.06	.98		3.0	113	4.23	2100	B10
Sand and Gravel	2.42		.77		2.9	124	3.78	2610	B4
					Slab Concrete				
Elgin Sand and Gravel	2.20		1.00	2.00	4.55	127	5.85	5375	A, B1-1, B2-1
Wayne Sand and Gravel	2.05			2.70(c)	4.25	126	5.00	4200	B1-2, B2-2, B2-3, B3, B4, B5-1, B6, B7, B8, B9, B10-1
"	3.10			4.05(c	6.37	126	5.00	2130	B5-2, B10-2

(a) Strength of joist concrete for series A, B1-1, B2-1 was 4470 p.s.i.
(b) Limestone screenings, 2 different gradings used.
(c) Wayne gravel aggregate was graded No. 4—3/4 in.

TABLE 2—RESULTS OF JOINT STRENGTH TESTS

Specimens were capped on bearing faces before testing. All values results of two tests except only one test was made for specimen 5.

Specimen No.	Ultimate Machine Load, lbs.	Ultimate Joint Strength p.s.i.	Ultimate Joint Strength lb. per lin. in.	Ratio Joint Strength to Total Allowable Shearing Stress on Web, v′ (a): $v = 180$, $v' = 225$	$v = 270$, $v' = 338$
Specimens with Concrete to Concrete Bond Plus Metal Tie as Indicated on Fig. 3.					
1	22,530	280	1,125	5.0	3.3
2	24,180	300	1,210	5.3	3.5
3	29,425	370	1,470	6.5	4.3
4	24,120	300	1,205	5.3	3.5
5	24,900	310	1,245	5.5	3.7
6	36,350	450	1,815	8.0	5.3
7	27,200	340	1,360	6.0	4.0
Specimens with No Concrete to Concrete Bond But with Metal Tie as Indicated in Fig. 3 (b)					
1′	11,135	140	555	2.4	1.6
2′	12,180	150	605	2.6	1.7
3′	10,855	135	540	2.4	1.6
4′	9,650	120	480	2.1	1.4
5′	14,675	180	730	3.2	2.1
6′	23,250	290	1,160	5.1	3.4
7′	3,500	40	175	0.7	0.5

(a) Based on 1¼ inch web and values recommended by A. C. I. Joint Code for $f'c = 3{,}000$ p.s.i. with plain and special anchorage of tensile steel.
(b) Concrete bond at joint prevented by heavily coating joist heads with paraffin-rosin mixture before casting on slab section.

shearing stress in the web. The total shear values of 225 and 338 are based on a 1¼ in. web thickness and the working stresses of 180 and 270 lb. per lin. in. for plain and special anchorage of the tension steel. These are the working shear stresses recommended in the Joint Code of the Institute for a 3,000 lb. concrete.

Specimens 1 to 7 gave joint strengths 5 to 8 times the allowable shear with plain anchorage and 3.3 to 5.3 times the shear allowed with special anchorage.

Specimens 1′ to 7 with joints having no concrete bond gave strengths ranging from 0.7 to 5.1 and 0.5 to 3.4 times the shear values allowed with plain and special anchorage respectively.

Conclusions Series A

1. Joint strengths produced with concrete bond ranged from 280 to 340 p.s.i. of bonding area.

2. Metal ties of any substantial type increase the joint strength remaining after fracture of the concrete bond.

3. Metal ties of the type used in specimen 6 may be expected to increase the ultimate joint strength above that obtained with concrete bond alone.

4. With the particular joist design investigated, the strength of joists with concrete bond may be expected to be 5 to 6 times the horizontal shear allowed with plain anchorage and 3.3 to 4 times the shear allowed with special anchorage.

SERIES B—TESTS OF LOAD CAPACITY AND PERFORMANCE OF FLOOR PANELS

Details of the floor test panels are given in Table 3 and Fig. 4 and 5. Table 1 gives data on the aggregates and concrete mixtures used.

Design and Construction of Panels

The reinforcement and details of assembly are shown by Fig. 6. Joist reinforcement design was based on the usual flexure formulas and the following factors and assumptions: satisfactory interaction between joist and slab: d = 8.37 in.; j = 0.875; f_s = 20,000 p.s.i.; f_v = 16,000 p.s.i.; v_c = 40 p.s.i. for panels 1-1 and 1-2, 60 p.s.i. for all other panels; live load of 40 lb. per sq. ft., dead load of 47 lb. per sq. ft. which includes a 16 lb. allowance for plaster ceiling and floor finish.

Joists were made in both wood and steel molds, the latter proving somewhat more satisfactory. Joists were erected on concrete masonry supports 21 days after manufacture. The formwork was supported from the joists. The slabs were cast and then covered with paper for three to five days. The forms were stripped in three to seven days depending on the need for them.

Deflection and strain gauge readings were made before and after casting the slab on the joists. The deflection at midspan averaged 0.123 in. and the observed stress in tensile steel, f_s, 5,990 p.s.i.

TABLE 3—DETAILS OF FLOOR TEST PANELS

Panels 5-1, 5-2, 10-1 and 10-2 were 14'-0" x 2'-0", single joist type. All other panels were 14'-0" x 4'-0", two joist type. Panel 11 composed of precast slab sections laid on joists in 1-3 portland cement mortar.

Panel No.	Joist Concrete: Type of Aggregate	Joist Concrete: Ave. Comp. Strength for Each Concrete. Time of Test p.s.i.	Ave. Comp. Strength of Slab Concrete p.s.i.	Type of Reinforcing Unit	Type of Joint Between Slab and Joist
1-1	Elgin sand and Haydite.	4470	5375	1	Concrete bond metal tie
1-2	"	3260	4200	1	"
2-1	"	4470	5375	2	"
2-2	"	3260	4200	2	"
2-3	"	3260	4200	2	"
3	Elgin sand and Cinders.	2780	4200	2	"
4	Elgin sand and Gravel.	2610	4200	2	"
5-1	Elgin Sand and Waylite.	1070	4200	2	"
5-2	"	1070	2130	2	"
6	Elgin Sand and Haydite.	3260	4200	4	Concrete bond. No metal tie.
7	Elgin Sand and Haydite.	3260	4200	4	"
8	"	3260	4200	2	Metal tie. No concrete bond.
9	"	3260	4200	3	Metal tie. Concrete bond.
10-1	Limestone	2100	4200	2	"
10-2	"	2100	2130	2	"
11	Elgin Sand and Haydite. (a)	3260	3260	4	Precast Slabs bonded to joist with mortar.

(a) Used for both joists and precast slab sections.

Test Procedure

The panels were tested 21 days after the slab was cast. Details of panel construction and joist reinforcement are shown in Fig. 4, 5 and 6. Initial deflection and strain gauge readings were made and loading of the panel with concrete block started. The block were uniformly distributed in separated tiers to prevent arching. Deflection and strain gauge readings were taken at load increments of 20 and 40 lb. The ultimate load was regarded as the load which produced collapse of one or both joists or a slow but evident failure as indicated by deflections.

Ames dials were placed as shown in Fig. 4 and deflections read to 0.001 in. The center dial readings were corrected to apply to midspan. The end dial readings compared so closely with the computed deflection of the joist at that point that they were disregarded.

Fig 7 shows panel 1-1 with a load of 160 lb. per sq. ft.

Observations of Performance of Panels During the Test

Fine tension cracks first developed in the lower head of the joists at the 20 and 40 lb. loads. Additional tension cracks formed up to about 120 lb. load after which there was mostly an extending and widening of cracks already started and the development of web cracking.

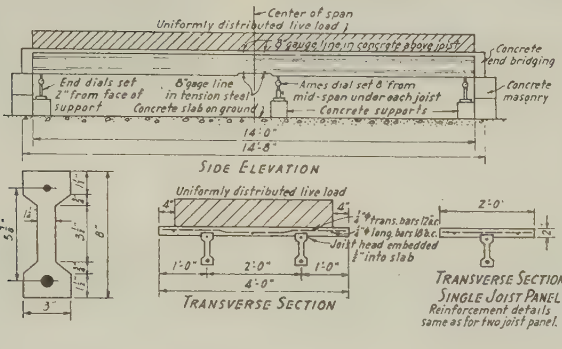


FIG. 4—DETAILS OF PANELS FOR FLOOR LOAD TESTS

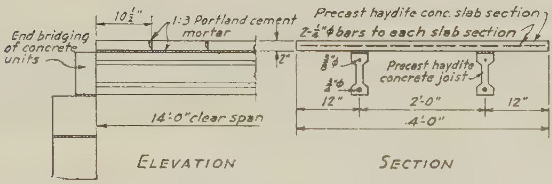


FIG. 5—DETAILS OF PANEL 11

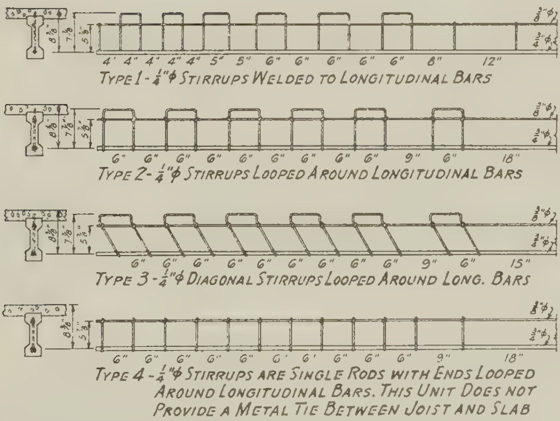


FIG. 6—REINFORCEMENT DETAILS OF JOISTS FOR FLOOR TEST PANELS

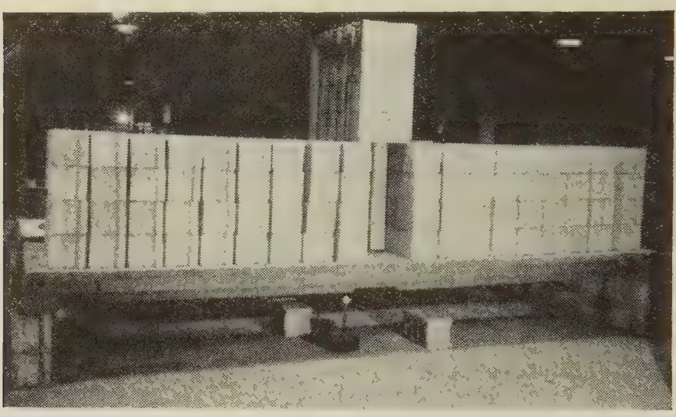


FIG. 7—PANEL B1-1 WITH LOAD OF 160 LB. PER SQ. FT.

Diagonal web cracking first appeared at 120 and 160 lb. loads, generally about 2 ft. from the supports. With load increase, new web cracks formed and existing ones extended diagonally or vertically, depending on their position in the span length. On reaching the junction of the web and top head, the cracks tended off horizontally although a few continued through the head to the slab. Cracking apparently was fully developed at loads of 200 or 240 pounds as further load increase caused very little change in the number or extent of cracks.

Excepting panels 2-2 and 8, it did not appear that the development of cracking from shear or diagonal tension was an important factor in the failure of the panels.

Observations at each increment of loading were made to detect any visible evidence of joint slippage. Slippage was noted only in panels 7 and 11. The joint of panel 7 had no metal tie or concrete bond. In the case of panel 11, which had precast slab members laid on top of the joists in mortar, mortar bond failed suddenly at 140 lb. load and the effect of joint slippage was quite evident as load was increased.

There was no visible indication of joint failure or slippage in any of the other panels tested including panel 8 which had metal ties but noconcrete bond.

Except in case of panel 2-2 which failed suddenly as a result of the failure of tension steel bond, all panels with monolithic slab and concrete bond failed in tension or tension steel bond or both. Failure occurred gradually, and generally was visible only by reference to the steady increase in deflection as shown by the dials. At the 280 lb. load the theoretical bond stress was 370 p.s.i. The measured f_s averaged 45,000 p.s.i. which, added to the initial stress, totals about 51,000 p.s.i. The manner of failure described above appears reasonable in view of these high stresses.

There was no evidence of compressive failure in the concrete of the joist or slab in any of the panels.

Discussion of Results

Table 4 gives the results as to load capacity, deflection at midspan with increments of loading and the measured f_s and f_c at the 160 lb. load. The table shows in addition the ratio of ultimate load to a live load of 85 lb. per sq. ft. which corresponds theoretically to f_s of 20,000 p.s.i. These ratios may be thought of as safety factors as applied to this particular joist design and A_s of 0.44 sq. in.

TABLE 4—RESULTS OF FLOOR LOAD TESTS

Panels uniformly loaded with concrete block arranged in separated tiers to prevent arching.
Load values refer to superimposed load, lb. per sq. ft.
Weight of floor, 31 to 33 lb. per sq. ft.
Ec lb. per sq. in., for slab concrete estimated at 4,000,000 excepting as follows: Panels 5-2 and 10-2, 2,700,000; Panels 1-1 and 2-1, 4,500,000; Panel 11, 2,000,000.

Panel No.	Ultimate Load, lb. per sq. ft.	Ratio of Ultimate Load to 85 lb. Live Load (a)	Mid-Span Deflections at Following Loads, lb. per sq. ft.; in.				Measured f_c and f_s at Load of 160 lb. per sq. ft.; lb. per sq. in.	
			40	80	160	240	f_c	f_s
1-1 (b)	160		0.093	0.169	0.384		900	25,700
1-2	320	3.7	0.067	0.152	0.367	0.590	650	26,000
2-1 (c)	160		0.060	0.147	0.425		860	27,600
2-2	240	2.8	0.096	0.209	0.483		1010	27,625
2-3	300	3.5	0.050	0.125	0.316	0.543	850	22,100
3	280	3.3	0.072	0.176	0.410	0.674	405	29,000
4	280	3.3	0.059	0.146	0.356	0.583	925	23,200
5-1	292	3.4	0.044	0.137	0.340	0.635	780	24,300
5-2	280	3.3	0.056	0.137	0.342	0.644	575	24,800
6	300	3.5	0.062	0.137	0.335	0.555	762	21,550
7	240	2.8	0.056	0.165	0.517	0.951	650	26,500
8	253	3.0	0.084	0.193	0.482	0.823	650	26,950
9	300	3.5	0.054	0.127	0.310	0.520	537	22,100
10-1	290	3.4	0.065	0.152	0.427	0.750	774	25,300
10-2	280	3.3	0.051	0.150	0.368	0.680	395	23,200
11	240	2.8	0.067	0.141	0.700	1.460	475	29,950

(a) Allowable live load based on f_s of 20,000 p.s.i.
(b) Not loaded to failure. Load of 160 lb. per sq. ft. left on panel 4 days increasing deflection 0.063 in. Recovery on removal of load was 0.343 in.
(c) Not loaded to failure. Panel sustained load of 160 lb. per sq. ft. for 7 weeks with no apparent distress.

Panels 1-1 and 2-1 were not loaded to failure. The 160 lb. load was maintained on panel 1-1 four days which resulted in increasing the deflection 0.063 in. On removal of the load, the panel recovered 0.343 in. of the 0.447 in. deflection. Panel 2-1 sustained the 160 lb. load seven weeks with no further visible weakening.

The ultimate loads for panels with monolithic slab and concrete bond ranged from a minimum of 240 lb. per sq. ft. with panel 2-2 to a maximum of 320 lb. per sq. ft. with panel 1-2. Of this group, panel 2-2 was the only one to fail at less than 280 lb. per sq. ft. The earlier failure of this panel was ascribed to a defective joist. The joist was cracked at one end along the bottom of the web and failure occurred by the lower head shearing off along the defect, thus destroying the tension bar bond and causing sudden collapse of the joist.

Tests on panels 7 and 8 were made principally for comparative purposes. Their ultimate loads were 240 lb. and 253 lb. per sq. ft. respectively. Panel 7 had no concrete bond or metal ties. Panel 8 had metal ties but no concrete bond.

Panel 11 was not of the usual type as the slab was a precast member. The mortar bond fractured at a load of about 140 lb. per sq. ft. causing excessive deflection but not complete failure. Panel sustained the ultimate load of 240 lb. per sq. ft. seven days with no further visible weakening.

For the group of panels with monolithic slab and concrete bond, the ratio of ultimate load to 85 lb. live load ranged from 2.8 to 3.7. Excluding panel 2-2, the minimum ratio is 3.3. Substantially higher ratios would be obtained by using the expected dead load and 40 lb. live load which is commonly accepted for residential structures.

The deflections may be studied from Table 4 or Fig. 8. Considering all panels tested, they ranged from 0.044 in. to 0.095 in. at 40 lb. load; 0.125 in. to 0.209 in.

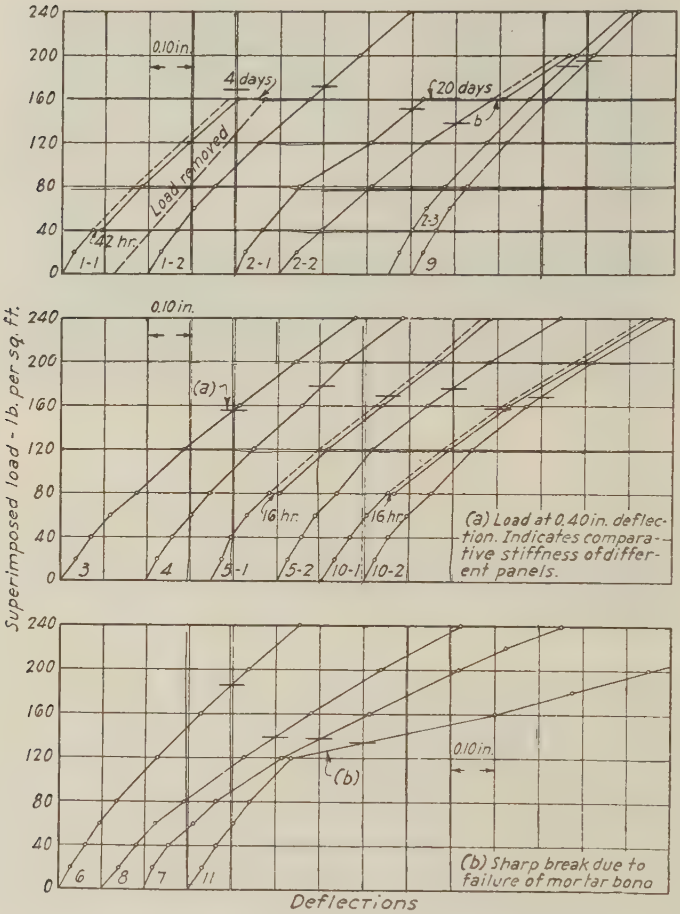


FIG. 8—LOAD DEFLECTION CURVES

at 80 lb. load; 0.310 in. to 0.70 in. at 160 lb. load; 0.52 in. to 1.46 in. at 240 lb. load. In Fig. 8 the points of 0.40 in. deflection indicated on each load-deflection curve show the relative stiffness of the various panels. In general, stiffness increased with load capacity, the main exception being 1-2 which deflected more than would be expected in view of its ultimate load. Panels 2-2, 7, 8 and 11 show relatively lower stiffness which is consistent with their lower load capacity. With all panels the deflection of 1/360 the span length required loads substantially greater than the maximum design load.

The values of f_c and f_s measured at the 160 lb. load tended to corroborate the load capacity and deflection values in indicating satisfactory interaction between slab and joist where connected with a concrete bond with or without metal ties. The

average of the f_c values compares closely with the theoretical stress of 746 p.s.i. The average of the measured f_s is considerably on the safe side of the theoretical stress of 28,400 p.s.i.

Effect of Type of Reinforcement

Panel 1-2 with welded reinforcement had the highest ultimate load capacity and panel 9 with diagonal stirrups showed the least deflection and somewhat less cracking. There appeared to be no great difference in performance or results with different types of reinforcements. However, it is reasonable that resistance to bond stress would be increased with the welded stirrup as compared with the stirrup with hooked connections.

Effect of Type of Aggregate Used in Joist Concrete

Results and performance of panels 2-1, 2-2, 2-3, 3, 4, 5-1 and 10-1 compared so closely as to indicate that type of aggregate in the joist concrete is not an important factor. In the case of the Waylite concrete joists, it is not believed that the cylinder strengths were indicative of the strength of concrete in the joist and particularly the concrete at the bottom of the joist on which tensile steel bond depends. This may be explained from the segregation in the wet mix used of the light Waylite aggregate, the heavier sand and cement particles tending to settle to the bottom of the joist. This segregation was noted in the cylinders tested. Therefore, the conclusion should not be drawn that strength of joist concrete is not a factor. It is believed that the joist concrete should have a minimum compressive strength of at least 2500 p.s.i. and preferably 3,000 lb.

Effect of Strength of Slab Concrete

Panels 5-1 and 10-1 had a slab concrete strength of 4200 p.s.i. The companion panels 5-2 and 10-2 had slabs of 2130 lb. concrete. Comparison of the results with these panels indicate that within these limits strength of slab concrete was not an important factor.

Effect of Type of Bond

The load capacity of panel 6 which had concrete bond but no metal tie was 300 lb. per sq. ft. indicating that an adequate joint strength is obtained with concrete bond alone. While metal ties of the type used may be regarded as desirable supplemental connections, their use did not increase the load capacity.

Conclusions Series B

1. The performance and results as to load capacity, deflection and measured stresses of panels with slab and joist connected with a concrete bond, with or without metal ties, indicated sufficient joint strength and interaction between slab and joist as to permit the use of the usual flexure formulas and allowable working stresses for T-beams in the design of floors of this type.

2. Panels with monolithic slab and concrete bond gave ratios of ultimate load capacity to maximum design live load of 85 lb. per sq. ft., ranging from 2.8 to 3.7.

3. With all panels tested, the deflection at design load was substantially less than 1/360 of the span length.

4. There appeared to be no great difference in performance or results with different types of reinforcements.

5. Results and performance of panels with joists made with different types of concrete compared so closely as to indicate the type of aggregate of the joist concrete is not an important factor.

6. For the range of conditions studied, strength of slab concrete had no marked effect on the ultimate load capacity of the floor construction.

7. While metal ties of the type used may be regarded as desirable supplemental connections, their use did not increase load capacity over that obtained with concrete bond alone.

For such discussion of this paper as may develop, readers are referred to the JOURNAL *for October, 1934 (Proceedings, Vol. 31). Discussions should be available to the Secretary by August 1, 1934.*

A Method of Evaluating Admixtures*

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In this discussion it is the authors' purpose to consider the general aspects of the question of admixtures in concrete, to explain the test methods followed in our more recent studies, and to describe a method of evaluating such materials.

GENERAL DISCUSSION OF THE FUNCTION OF ADMIXTURES

A rather definite concept of the function of admixtures and their effect on the properties of concrete has resulted from extensive laboratory studies of many materials and from field experience with some of them. It can be stated as a general rule that an increase in the necessary water content of the cementing paste in concrete is detrimental. It follows, therefore, that any beneficial effect which an admixture may have must compete with the detrimental effect of any increase in the water content which its use may incur. This holds for portland cement as well as for other powdered materials. For example, to add cement only to a mix will increase the strength to a greater degree than to add the same amount of cement together with water. The fact that the addition of cement plus water may show an increase in strength merely illustrates that the detrimental effect of the added water did not offset the beneficial effect of the added cement.

Many admixtures exhibit some beneficial effect which tends to offset to a greater or less degree the ill effects of increases in water content which their use may bring about. A given admixture may be beneficial in one or more of the following ways:

1. It may improve the texture of the concrete mix, a physical effect.
2. It may have cementitious properties of its own.
3. It may be puzzolanic.

Physical Effects—Nearly all of the powdered admixtures, as well as some of the other types, may be beneficial through their physical effect on the mix. This physical effect consists in a stiffening of the cement paste which tends to prevent its stratification while the concrete is

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still in the plastic condition. When a paste is originally of a watery character, stratification after agitation of the mix has ceased and before hardening has taken place is a natural consequence; the heavier particles of cement settle away from the under surfaces of the aggregate, leaving only the water and the finest cement particles in immediate contact. That such stratification exists is shown by the fact that at the plane of fracture in concrete where wet mixtures were used, the aggregate will be found adhering to the upper fragment (as placed) leaving the cavities in the lower.

Because of this unequal bond the full value of the cement is not realized. By stiffening the cement paste, by the addition of more cement, the reduction of water content, the addition of fine powder, or by any other means not in itself detrimental to the chemical reactions, stratification is minimized and strength, texture, and watertightness improved.

It can be seen from the discussion immediately above, as well as from that of workability which follows, that the physical effectiveness of an admixture will depend to an important degree upon the character of the original mix. Naturally, if the cementing paste is already homogeneous and plastic, a further increase in fine material will not show any benefit, but will probably require the addition of water which may cause loss of strength. In mixes in which the cement paste is of a watery character (high water-cement ratio) some quantity of fine material can usually be added without the addition of water, with a general improvement in all properties of the mix. In fact, in certain mixes the improvement may be sufficient to permit some increase in the water content without loss of strength or watertightness.

An increase in water content, which commonly attends the use of an admixture, may be expected to increase shrinkage since shrinkage is largely due to loss of uncombined water from the paste. Likewise, any increase in the proportion of paste gives greater opportunity for volume change.

Cementitious Admixtures—Some admixtures show beneficial effects by virtue of cementitious properties of their own. Those materials of this class which were included in these studies gave some strength to concrete even when used to the complete exclusion of portland cement. When used in combination with portland cement the results usually showed the concrete to have a strength just about equal to the strength normally produced by the amount of cement used plus the strength contributed by the hardening of the admixture itself. In the leanest mixes some evidence of the physical effect in addition to a cementitious effect could also be detected.

Puzzolanas.—Some admixtures have no cementitious value of their own, but react with the calcium hydroxide which is liberated during the hardening of portland cement, to form compounds which add to the strength of the mix. Such materials are commonly called puzzolanas. They too may have beneficial physical effects in certain mixes.

Justification for the Use of Admixtures. These beneficial effects have been found singly and together by various investigators in a great variety of materials. It does not necessarily follow that because a material shows a definite benefit to one property or another, its use in concrete is thereby justified. Study has shown that when beneficial effects are found with a given material, its use should be considered only as *one* way to secure that end, and decisions for or against its use should be based not only on the mere fact of its effectiveness, but also upon the cost of producing the desired results as compared with the cost of similar results by some other method.

ESSENTIALS OF A STUDY OF ADMIXTURES

Variables Involved. From the foregoing considerations it is clear that an adequate study of admixtures must include their use in a wide range of mixes, for their effectiveness varies with the character of the mix. Also, such a study must cover a wide range of proportions of admixture, for it is apparent that though one proportion may be detrimental, a different proportion of the same material might have the opposite effect. For materials that are cementitious or puzzolanic, it is necessary to consider the results after different periods of moist curing, for beneficial effects from such materials may slowly develop if moist curing is prolonged. Finally, since there are many materials capable of producing similar results, any one material must be considered in comparison with others.

This latter requirement, the comparison of different materials, is the one most difficult to meet in a study of this kind, particularly when workability is the basis of comparison. Since admixtures are usually sold to improve workability, it is important in a study of this kind to set up some rational basis of comparison whereby the relative workabilities of different mixes may be judged. In this study, about one year was spent on this phase of the problem.

General Discussion of Workability. The results of that work may be found in the JOURNAL of the American Concrete Institute for February and September, 1932,[1] in a paper discussing the problem of workability of concrete in general with brief reference to the effect of admixtures. It was found that workability may be conveniently

[1] "Studies of Workability of Concrete," and subsequent discussion, by T. C. Powers.

studied and discussed by arbitrarily considering the mix to be made up of two parts: the aggregate, and a paste composed of cement and water. If an admixture is present, it is considered as part of the paste. For specific materials, workability was found to depend on three factors, viz:

(a) The quantity of paste per unit volume of concrete.

(b) The consistency of the paste.

(c) The gradation of the aggregate.

The first two of these factors are of particular interest in the study of admixtures.

In considering the paste in connection with workability of concrete, it should be remembered that the minimum requirement for workability is that the gradation and quantity of the aggregate must be such that it will be suspended by the enveloping paste. The weight of the heavy aggregate can be carried by the fluid or semi-fluid paste only when it is distributed over a sufficiently large surface. And not only must the surface area be sufficiently large, but also the various sizes of particles must be proportioned so that the body of paste is spread into a continuous relatively thin film.

Pastes made with different quantities of a given solid, as for example, cement, have varying degrees of fluidity. They may be likened to lubricating oils of different viscosities. A light oil may carry a given load without breaking down, but the greater the load, the "heavier" must the oil be, other factors being equal. Similarly, a light cement paste can carry a given load of aggregate only when that load is spread over a sufficiently large surface area. If the load is too great, or the surface area is too small, plasticity is poor or entirely lacking.

When a mixture lacks workability it may be possible to bring about the necessary improvement by increasing the proportion of fine material in the paste, thus giving it more "body" and supporting power. Such a change may make the concrete as a whole more mobile even though the paste is stiffened. On the other hand, if the paste has sufficient supporting power to carry the load of aggregate, to stiffen the paste by adding fine solids may besult only in stiffening concrete as a whole.

With the characteristics of the paste and the aggregate such as to give plasticity, profound changes in workability may be effected by changes in the quantity of paste. A dry, stiff mix may be changed to a wet, easy flowing mix merely by increasing the proportion of paste. Thus, any change in a paste which results in an increase in the volume of the paste, such as the addition of solids, or water, or both, tends to increase the mobility of the concrete.

From these statements it is clear that adding to the solid content of the paste and thereby giving it a stiffer consistency, affects the mobility of concrete as a whole according to original character of the paste and of the aggregate. To some mixes, a powder may be added without increasing the water content and without decreasing the mobility of the mix. In other cases, additions of a powder alone will so stiffen the paste as to stiffen the mix as a whole and require the addition of water to restore mobility. Thus, from the standpoint of workability, a powder itself (*not* powder plus water) may or may not be beneficial according to the original character of the mix. To add both powder and water generally improves workability through the increase in paste content. From this angle it is again seen that a comprehensive study of admixtures must include their use in a wide range of mixes.

One of the major conclusions reached in the preliminary studies was that workability of concrete is determined to a much greater extent by the *quantity* and *consistency* of the paste than by the materials of which the paste is composed. With the quantity and consistency of the paste held within narrow limits, the various substances tried gave only minor differences in workability. Stated more specifically:

Any of the powdered materials tested, or blends of two materials, when used in such proportions as to produce a paste of given volume and consistency, gave when combined with the same kind and grading of aggregate, concrete mixes of substantially equal workability.

An understanding of this important conclusion should prevent the confusion which often exists as to the value of additions of powdered materials to concrete mixtures. Any comparative analysis of concrete with and without admixture which fails to take into account differences in paste content or consistency is likely to result in unsound conclusions.

BASIS OF DESIGN OF MIXES

Because comparisons on the basis of strength or other physical properties of the hardened concrete are comparatively simple, the mixes in this study were designed primarily to afford comparisons on the basis of workability. Since there is no absolute measure of this property, the procedure followed was to make up mixtures of plain concrete as standards of comparison and to make, as nearly as could be done, all of the mixes containing admixtures identical with one of these standards. For this purpose the foregoing generalization from the preliminary tests was accepted as a basis of computing the materials

required in the first approximate mixes. The final mixtures were adjusted to give as accurately as possible identical workability among the specimens to be compared. The method followed is described below.

Pastes of the Same Consistency. The proportion of a material required to produce a paste of given consistency was determined by trial, using as a measure of consistency the rate of flow of the paste through a tube, under a pressure of 1 p.s.i. For example, a cement paste of 5.1 gal. of water per sack of cement (about 41 per cent solids by absolute volume, 67 per cent by weight) had, in the particular apparatus used, a flow-rate of 10 cc. per second. The percentages (by weight and absolute volume) of other solids required to produce pastes having the same flow-rate are given below:

Admixture	Per Cent of Solids in Paste	
	Abs. Vol.	Weight
Bentonite (Aquagel)	3	7
Diatomaceous earth (Celite)	12	21
Hydrated lime	24	45
Hydraulic lime (Flamingo)	32	58
Magnolia cement	37	63
Crystalline talc	31	52
Pumicite from Kansas	32	53
Tripoli silica (Barnsdall)	33	56
Pumicite from California	34	57
Colloy	44	67
Mississippi loess	40	64
Pulverized blast-furnace slag	45	70

Similarly, for portland cement pastes of other water contents, the per cent of solids required to give paste of the same flow rate was determined by trial for the various admixtures.

Details of the Method of Mix Design. To avoid extending the work too greatly the tests were confined to an intermediate consistency (slump 3 to 5 in., remolding effort 35) and to 3 basic mixes having water contents of about 5, 6, and 7½ gal. per sack. The exact composition of the three mixes chosen are as given in Table 1.

It will be noted that the three mixes, A. B, and C, are all shown to have the same mobility, as represented by the remolding effort of 35, and slumps falling within a narrow range. It should not be inferred from this that they have equal workability. These mixes differ in several significant respects to such an extent that each represents a different degree of workability.

One of the most important differences between the three mixes is in the quantities and consistencies of the pastes which they contain, with the resultant differences in cohesiveness of the concrete. Mix A, for

TABLE 1—BASIC MIXES USED IN STUDY OF ADMIXTURES

Fine and coarse aggregate from Elgin, Ill.

Coarse aggregate graded:
25%—No. 4- ⅜in.
25%—⅜-¾ in.
50%—¾-1½ in.

Fine aggregate graded:
0-No. 4; f.m. 2.92; gradation controlled by proportioning from separated sizes.

	1	2	3	4	5	6	7	8	9	10
			Cement Content		Water Content					
Ref. No.	Mix by Weight	% Sand by Weight	Sacks per Cu. Yd.	Lb. per Cu. Yd.	Gal. per Sack	Gal. per Yd.	Paste Content % Absol. Volume	Slump in.	Remolding Effort *	Flow Rate of Paste cc. per Sec.
A	1:1.93:3.23	37.5	6.47	608	5.1	33.0	27.8	3	35	10.0
B	1:2.50:3.94	39.0	5.32	500	6.2	33.0	25.7	4	35	14.0
C	1:3.30:4.70	41.0	4.37	420	7.7	33.7	24.5	5	35	15.5

* Studies of Workability of Concrete; JOURNAL Amer. Concrete Inst., Feb. 1932, *Proceedings* v. 28, p. 419.

example, is made with a paste which had a flow-rate of 10 cc. per sec., while mix C contained a considerably "lighter" paste, flow-rate 15.5 cc. per sec. Because of the differences in the consistencies of the pastes there were also differences in the quantity of paste required, as shown in column 7. When using a "light" paste, it is necessary for the weight of the aggregate to be distributed over a larger surface area; hence, as shown in column 2, mixes B and C contain higher percentages of sand than mix A.

Thus it is seen that although the three mixtures have the same degree of mobility, they differ as to gradation of aggregate and quantity and consistency of the paste. In other words, the differences are those between lean and rich mixes.

Using these three separate standards of workability, other mixes were designed using less cement than the standard, but with sufficient admixture to give the same consistency and total quantity of paste. Each mixture was then compared with its corresponding standard and the necessary adjustments made in quantity of water and admixture to give the same remolding effort and the same total quantity of paste. The procedure followed in designing these mixes can best be illustrated by an example:

Example

Required: a concrete mix having a workability equal to mix A, but containing only 400 lb. cement per cu. yd., an admixture being added to give the required workability.

Solution

Total paste in Mix A = 27.8% = 7.50 cu. ft. per cu. yd. abs. vol.
This is the amount of paste required in the new mix.

The cement portion of the paste in the new mix must have the same consistency as in Mix A, hence will be made up as follows:

400 lb. cement $= \left(\dfrac{400}{62.4 \times 3.15}\right)$ .... = 2.04 cu. ft. abs. vol.

water at 5.1 gal. per sack $\left(\dfrac{5.1 \times 400}{94}\right)$ = 21.7 gal........... = 2.90 cu. ft. abs. vol.

Total volume of cement paste............................... = 4.94 cu. ft. abs. vol.

7.50 — 4.94 = 2.56 cu. ft. to be made up by admixture paste, which also must have the same consistency as the paste in Mix A.

This admixture paste must also have the same consistency as the paste in Mix A. The next step is to determine the proper proportion of admixture to meet this requirement. Assume for this example that for a flow rate of 10 cc. per sec., the admixture paste must have 40% solids by absolute volume and that the admixture itself has a specific gravity of 2.62.

Thus, 2.56 x .40 = 1.02 cu. ft. admixture absolute volume. This amount of admixture =

2.62 x 62.4 x 1.02 cu. ft. = 167 lb.

2.56 — 1.02 = 1.54 cu. ft. water required = 11.5 gal.

The final composition of the paste, therefore, is as follows:

Cement	— 400 lb.	= 2.04	
Water	— 21.7 gal.	= 2.90	
			4.94 cu. ft. cement paste
Admixture	— 167 lb.	= 1.02	
Water	— 11.5 gal.	= 1.54	
			2.56 cu. ft. admixture paste
			7.50 cu. ft. total paste

The aggregate combination will be the same as in Mix A.

Checking by Trial

Such a computation was generally taken merely as the first approximation of the correct quantity. The final mix was determined by trial, using the remolding test, but in general ending with the same volume of paste as in the standard mix.

A typical illustration of the trial method is given in Fig. 1. This diagram gives the data for determining three percentages of a particular admixture required to give three leaner mixes having workability equal to that of Mix A of Table 1. In this method a mixture computed as described above would be tested with at least two percentages of admixture, one above and one below the computed value. Each of the two mixtures would then be tested in the remolding apparatus with varying proportions of water.

For example, the upper left hand diagram of Fig. 1 gives the result for one such mixture tested with two percentages of admixture in the manner described, each with several percentages of water. The remolding effort is shown in relation to the paste content, there being a curve for each of the two percentages of admixture.

The next step in this method is shown in the lower diagrams. The lower left, for example, is derived directly from the diagram above. It shows remolding effort in relation to per cent of admixture for the theoretically correct proportion of paste,—that of Mix A (27.8 per cent). At the same remolding effort as for Mix A, 35 jigs

the intersection of the curve indicates the necessary proportion of admixture, 15 per cent. The computed value was 16 per cent as indicated.

With the theoretically correct quantity of paste in the mix, the percentage of admixture required to give the right mobility did not always agree perfectly with the computed quantity, but the discrepancies were not large as shown in Table 2.

TABLE 2—PASTE AND ADMIXTURE CONTENT FOR MIXES HAVING SAME REMOLDING EFFORT AS STANDARD MIX A

Admixture	Cement Content Sacks per Yd.	Computed		Trial Method	
		Paste Content %	% Admix. by Weight of Cement	Paste Content %	% Admix. by Weight of Cement
Bentonite (Aquagel)	5.2	27.8	1.3	27.8	1.2
	4.2	27.8	2.9	27.8	2.8
	3.6	27.8	4.3	27.8	4.3
Diatomaceous Earth (Celite)	5.2	27.8	4.3	27.0	3.5
	4.2	27.8	9.5	27.0	10.0
	3.6	27.8	13.7	28.0	18.0
Hydrated Lime	5.2	27.8	11.4	27.5	12.0
	4.2	27.8	25.4	27.8	30.0
	3.6	27.8	36.7	28.0	42.0
Hydraulic Lime (Flamingo)	5.2	27.8	17.0	27.8	18.0
	4.2	27.8	38.1	27.7	38.0
	3.6	27.8	55.0	27.9	62.0
Crystalline Talc	5.2	27.8	14.0	——	——
	4.2	27.8	30.0	——	——
	3.6	27.8	44.0	——	——
Kansas Pumicite	5.1	27.8	14.2	28.2	14.0
	4.2	27.8	31.4	27.9	30.0
	3.6	27.8	45.5	27.9	42.0
Tripoli Silica (Barnsdall)	5.2	27.8	15.0	27.9	14.0
	4.2	27.8	35.0	27.9	32.0
	3.6	27.8	50.0	27.9	48.0
California Pumicite	5.2	27.8	16.0	27.8	15.0
	4.2	27.8	34.0	27.8	35.0
	3.6	27.8	50.0	27.8	51.0

In general, the computed and trial values agree remarkably well, especially in view of the testing difficulties involved. The greatest disagreement was found with diatomaceous earth. With 27.8 per cent paste and the computed percentage of admixture the remolding effort was less than 35; that is, the mix was too wet. The mix could have been stiffened the required degree by raising the percentage of admixture or by lowering the percentage of water. The latter course was followed, as shown by the lowered paste content, to give the admixture the benefit of any doubt, for the less admixture required, the better it appears in comparison with other materials, as far as workability is concerned.

Factors Common to Mixes of Equal Workability. Mixtures designed for constant workability by the above procedure have the following characteristics in common:

1. The same mobility as determined by the remolding test.
2. Slumps falling in a narrow range within which, however, some admixtures gave consistently greater slumps than others at the remolding effort of 35.
3. The same aggregate gradation.
4. Approximately equal paste content.
5. Approximately equal paste consistencies.

6. Equal mortar-voids ratio.[1]
7. Constant value of b/b_o.[1]
8. Constant paste-voids ratio.

The principal variance between mixes so designed when made with different materials was in the degree of cohesiveness. Since no method of measuring or evaluating cohesiveness in concrete mixtures has been developed there must remain some inexactness in the comparison of these mixtures on the basis of true workability. It can be stated, however, that when mixtures have all the above factors in common, the differences in cohesiveness are negligible from the practical standpoint of the placeability of the concrete. In the opinion of the writers, these small differences certainly have little or no economic value. Any of the mixtures designed by this method for equal workability would serve equally well on a given job, except perhaps under conditions which require transportation over long distances without agitation en route. In such cases those admixtures which minimize the segregation of water might retard packing of the concrete during transportation.

There is one important feature in this method of designing mixes that is likely to be overlooked—the method of selecting the proportion of sand. It may be noted in Table 1 that the proportion of sand in the three standard mixes is different in each case. The percentage used is the optimum as determined by trial[2]; it is that which is best suited to the quantity and consistency of paste in the mix. When admixture paste is substituted for cement paste the aggregate grading remains unchanged because the paste volume and consistency are also unchanged. Thus, mixtures lean in cement but containing admixture have aggregates graded to suit a richer, straight portland cement mix of equal paste content. When the usual practice is followed,—simply adding the admixture and water to a given mix—the grading, if it was correct for the original mix, is no longer correct after adding the admixture; it is over-sanded and hence does not give the maximum result for that quantity of admixture.

Importance of Aggregate Gradation. In all of the studies of workability the effect of aggregate gradation is so persistently in evidence that a discussion of this subject cannot be closed without at least mentioning the effect of gradation lest the paste factor receive undue emphasis. That workability may be materially improved by the use of more cement or an admixture together with water is general knowledge. However, lack of workability is probably more often due to

[1]Bull. 137, University of Illinois Engineering Experiment Station.

[2]"Studies of Workability of Concrete.", Journal, Am. Concrete Inst., Feb. 1932, *Proceedings*, v. 28, p. 429.

grading faults than to any other one factor, and to attempt to correct a mixture by means of changes in the paste is often needlessly expensive and may not give the desired results. Therefore, when more workability is needed, use of additional cement or an admixture would seem to be a last resort to be considered only after the best possible combination of available aggregates has been found inadequate or too expensive.

EVALUATING AN ADMIXTURE FROM TEST RESULTS

The real worth of an admixture should be determined on the basis of its effectiveness in producing, under definite curing conditions, the specific properties desired in concrete. This may require data on its effect on strength, workability, volume change, resistance to disintegration by freezing and thawing, and resistance to the action of strong sulfate waters.

Whichever of these properties is considered of primary importance, there are known to be several ways of effecting the desired results; hence, the use of an admixture should be considered as only one of the possible means that should be compared. For example, if it is desired to bring about an improvement in workability, that may be done be enriching the mix, by correction of aggregate gradation, or by the use of almost any one of a host of available finely powdered materials.

Again, if an admixture is found to improve strength, it should be remembered that there are other admixtures having similar effects, and that an equal increase could be effected by any of at least four possible changes in the mix: changes in consistency, richness, gradation or size of aggregate. Accurate evaluation of an admixture requires that its cost be compared with that of other materials or methods which also give the desired results.

The method of evaluation used in these studies will be illustrated on the basis of strength and workability data. It is first necessary, of course, to have the test data from a series of mixes designed according to some definite scheme. The test data are then reduced to convenient terms by means of interpolation curves and finally, for the purpose under discussion, reduced to the form shown in Fig. 2, 3, 4, and 5. The solid curves in each diagram show, for the respective ages at test, the relations between cement content and admixture content for various strengths of concrete. Consider for example, the middle diagram of the upper row of Fig. 2 which represents the 28-day results from Barnsdall. Points on any one of the solid curves represent mixes containing combinations of cement and admixture which produced the strength indicated. Each curve terminates at a point on the ordinate through zero representing a straight portland cement mixture.

Such curves make it very simple to determine the value of an admixture in producing concrete of given strength. A certain point on the curve marked 3000 in the diagram above referred to, for example, shows that 3000-lb. concrete can be produced with 400 lb. of cement in combination with 40 lb. of admixture, or by 430 lb of cement alone. In this case, it takes the 40 lb. of admixture to do the work of 430 — 400 = 30 lb. of cement. For this purpose, it is plain that a pound of admixture is as effective as 30/40 = .75 lb. of cement and it, therefore, has the same proportional value as far as strength is concerned.

From the fact that the several curves are neither straight nor parallel, it is obvious that the value of the admixture, relative to that of the cement, is different for other combinations of cement and admixture.

With a chart such as this for each admixture under consideration, comparisons between the admixtures as well as between cement and admixture can be made conveniently.

The broken lines, A, B, and C, of these charts connect points representing mixes having workabilities comparable with standard mixes, A, B, and C, respectively. On the middle Barnsdall diagram, for example, reading from the top downward along line B, there may be ascertained the quantities of admixture which were required to maintain constant workability as the cement content was reduced. A mix containing 80 lb. of admixture plus 375 lb. of cement, for example, is shown to have the same workability as mix B, which contains 500 lb. of cement. Or, 80 lb. of admixture is as effective as 500 − 375 = 125 lb. of cement. Thus, one pound of admixture is worth 125/80 = 1.56 lb. of cement as far as workability alone is concerned. The difference between the strengths of the two mixes under discussion should not be overlooked, however, 2800 for the mix containing admixture, and 4000 for the plain mix of equal workability.

From the fact that these broken lines are straight and parallel, it is apparent that the above ratio, 1.56, is general for this admixture when used with this particular set of materials. That ratio, therefore, indicates for these materials the relative worth of this admixture as a workability agent.

It is believed that this study of methods is far more important than the specific data on various admixtures presented below. From the foregoing studies it is to be expected that the results obtained with any specific series of tests will depend to a considerable degree upon the test procedure and combination of materials used. Experience

has shown that the results obtained are affected considerably by the gradation of the sand. Also, an admixture behaves differently according to the richness of the mix, condition of curing, and age at test. Furthermore, the quantity of admixture itself is a factor.

RESULTS OBTAINED WITH DIFFERENT ADMIXTURES

Compressive Strength. It can be seen that no blanket statement as to the worth of any admixture not definitely detrimental would be valid. Too much depends on the conditions discussed above. However, it will be of interest to examine the results obtained under the specific conditions described in this report. These results may not represent the exact worth of a given admixture for any particular job, but they should be fairly representative of the comparative behaviors of the different materials.

The 28-day charts of Fig. 2, 3, 4, and 5 have been analyzed by the method described in the above section. The results are summarized in Table 3 which shows indices of relative worth for each admixture on the basis of 28-day strengths. Four values for each admixture are given, two for results obtained in lean mixes and two for richer mixes (400 and 500 lb. cement plus admixture, respectively). Under each class of mix, two indices are given, one which holds for quantities of admixture ranging from 0 to the maximum quantity which could be represented by a single index with no appreciable error, and the other for a quantity double that maximum.

In the last column indices determined on the basis of workability are given. It should be kept clearly in mind that in this column the values are determined entirely on the basis of the relative quantities required to produce a given effect on workability with no weight at all given to differences in other properties of the concrete.

Freezing and Thawing Tests. Freezing and thawing tests were made on concretes containing four of the admixtures: Celite, California pumicite, hydrated lime, and hydraulic lime. The specimens were exposed to the first freezing after 28 days moist curing. The results are given in Fig. 6 which shows the relation between the number of cycles required to cause complete disintegration of 6 by 2-in. discs and the 28-day compressive strength of cylinders made from the same batches. All of the mixes represented on this diagram had the same workability except those marked with crosses, which were plain mixes having lower standards of workability. The scattering of the points from a single curve is well within the probable error of tests of this type, indicating that mixes of equal strength had substantially equal resistance to disintegration.

In general, the points representing concretes containing admixtures lie above those for plain mixes, but the difference is not great enough to indicate important differences in durability. Those plain mixes represented by crosses were of considerably poorer workability than those containing admixtures. Such mixtures have relatively poor texture which may possibly reduce durability. The one plain mix which had workability equal to the admixture concretes was that represented by the circled cross. This mix was the standard of workability for the group. While this point falls below the curve, it represents the average of 5 discs, some of which were more durable than any of the specimens containing admixtures. The points representing admixtures are based on two discs only.

In view of the foregoing, it appears that the data warrant the conclusion that the 28-day strength is a fair criterion of probable durability regardless of the presence or absence of an admixture.

A similar plotting on the basis of the 6-mo. strengths gave a somewhat better alignment of points than in Fig. 6, indicating that the later strength would be an even better criterion of durability. The freezing and thawing specimens, it must be remembered, were first exposed at 28 days. As the thawing was carried out in warm water, the hardening process was free to continue (probably at a reduced rate) throughout the period of exposure which in some cases was about a year. Under these circumstances the better index afforded by the 6-mo. strength is not surprising.

Volume Change. Measurements of change in length due to drying were made on concrete prisms made of the same mixes as used in the freezing and thawing tests. The results for mixes with and without admixture are shown in Fig. 7 which gives the relation between compressive strength and shrinkage at the end of 6 mo. in air at 50 per cent humidity (following 7 days moist curing). The date do not warrant a general conclusion because of their restricted scope. However, they indicate that except for Celite and the larger quantities of California pumicite, the differences in shrinkage between concretes of the same strength with and without the admixtures were not great.

Table 3 and Fig. 1—7 of the foregoing paper appear on the following pages

For such discussion of this paper as may develop, readers are referred to the JOURNAL *for October, 1934 (Proceedings, Vol. 31). Discussions should be available to the Secretary by August 1, 1934.*

TABLE 3—INDICES OF RELATIVE WORTH ON THE BASIS OF 28-DAY STRENGTHS AND OF WORKABILITY FOR ADMIXTURES USED IN COMBINATIONS INDICATED IN THE TABLE

Name of Admixture	Indices on Strength Basis — Total Quantity of Cement and Admixture, Lb. per Cu. Yd. — 400		500		Indices on Workability Basis
	Index	Quantity of Admix.	Index	Quantity of Admix.	
Tripoli Silica (Barnsdall)	0.70	0–40	0.37	0–40	
	0.50	80	0.20	80	1.56
California Pumicite	0.80	0–50	0.35	0–50	
	0.60	100	0.25	100	1.47
*Pulverized Blast-Furnace Slag	0.60	0–50	0.20	0–50	
	0.50	100	0.15	100	1.07
Hydraulic Lime	0.90	0 50	0.50	0–50	
	0.75	100	0.40	100	1.37
Hydrated Lime	0.75	0–40	0.12	0–40	
	0.50	80	0.12	80	1.76
Magnolia Cement	1.00	0–50	1.00	0–50	
	0.85	100	0.85	100	1.20
Diatomaceous Earth (Celite)	3.50	0–10	1.50	0–10	
	2.40	20	1.25	20	5.00
Colloy (Precipitator Ash, Lot No. 11893)	0.90	0–50	0.90	0–50	
	0.80	100	0.80	100	1.00
Precipitator Ash (Lot No. 11897)	0.80	0–50	0.70	0–50	
	0.65	100	0.50	100	1.11
Bentonite (Aquagel)	0.00	0–4	-5.0	0–4	
	–0.65	8	-5.0	8	17.60
Crystalline Talc	0.65	0–40	0.25	0–40	
	0.35	75	–0.25	80	1.67
Kansas Pumicite	0.15	0–40	0.25	0–40	
	0.12	80	——	80	1.92

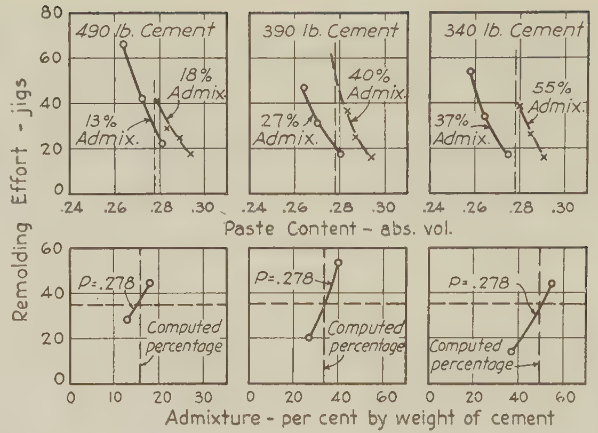


FIG. 1—ILLUSTRATION OF TRIAL METHOD OF DETERMINING THE PERCENTAGE OF ADMIXTURE REQUIRED TO GIVE WORKABILITY EQUAL TO STANDARD MIX "A"

NOTE TABLE 3

Negative values for the index indicate that extra cement is required when the admixture is used.

*The effectiveness of slag may be expected to vary with its fineness.

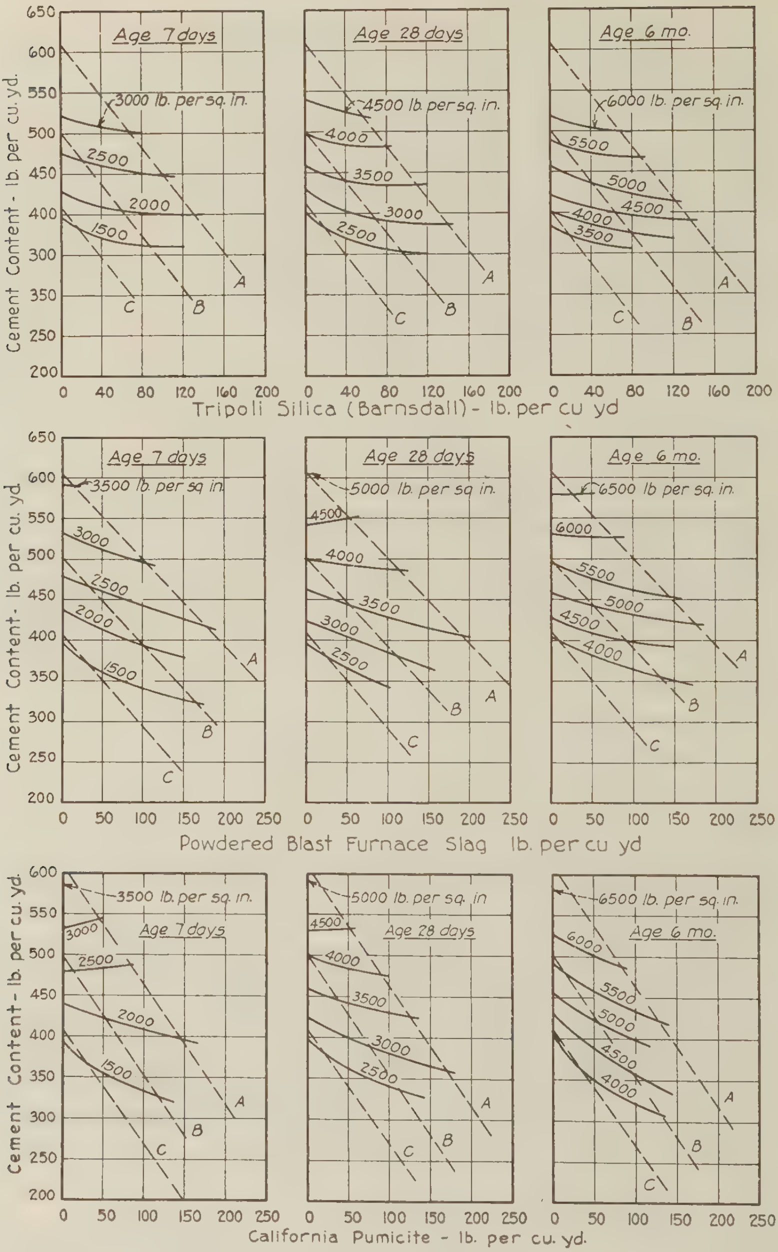


FIG. 2—EFFECT OF ADMIXTURES ON CEMENT REQUIREMENTS OF CONCRETE

Broken lines connect points of equal workability.

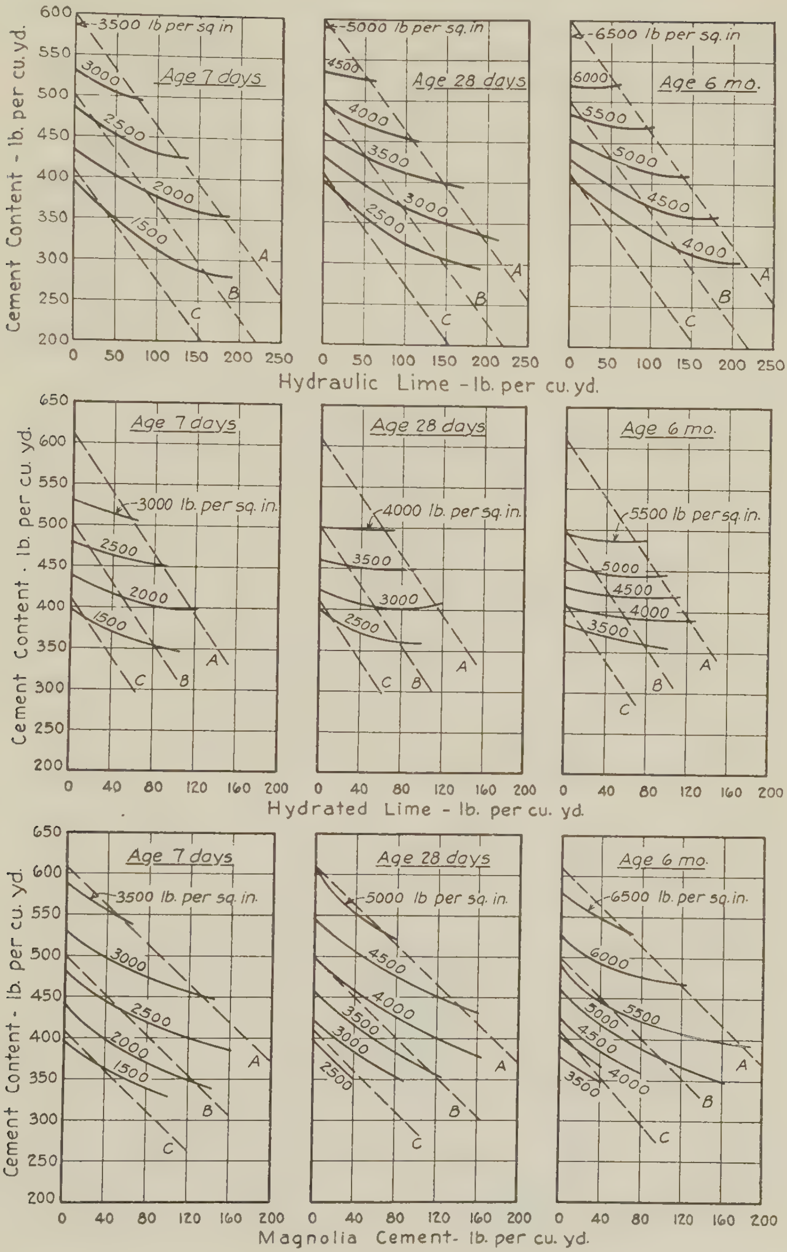


FIG. 3—EFFECT OF ADMIXTURES ON CEMENT REQUIREMENTS OF CONCRETE

Broken lines connect points of equal workability.

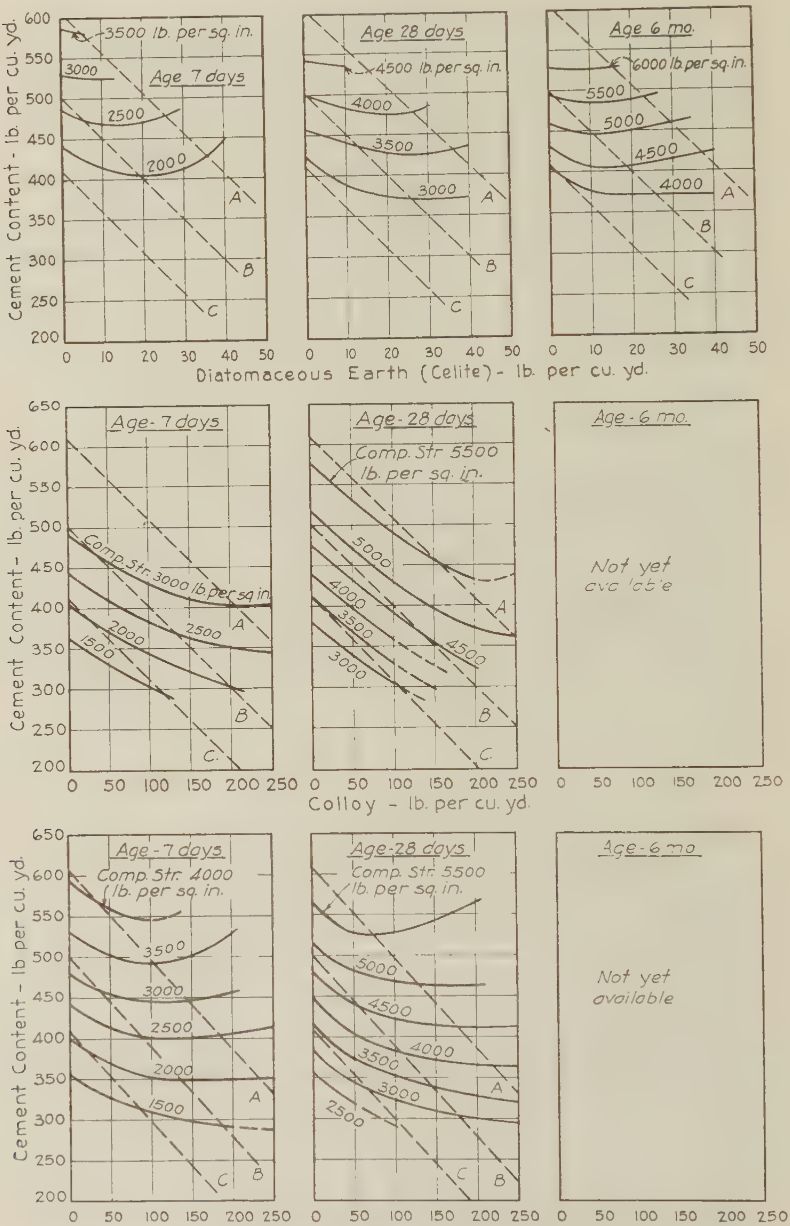


FIG. 4—EFFECT OF ADMIXTURES ON CEMENT REQUIREMENTS OF CONCRETE

Broken lines connect points of equal workability.

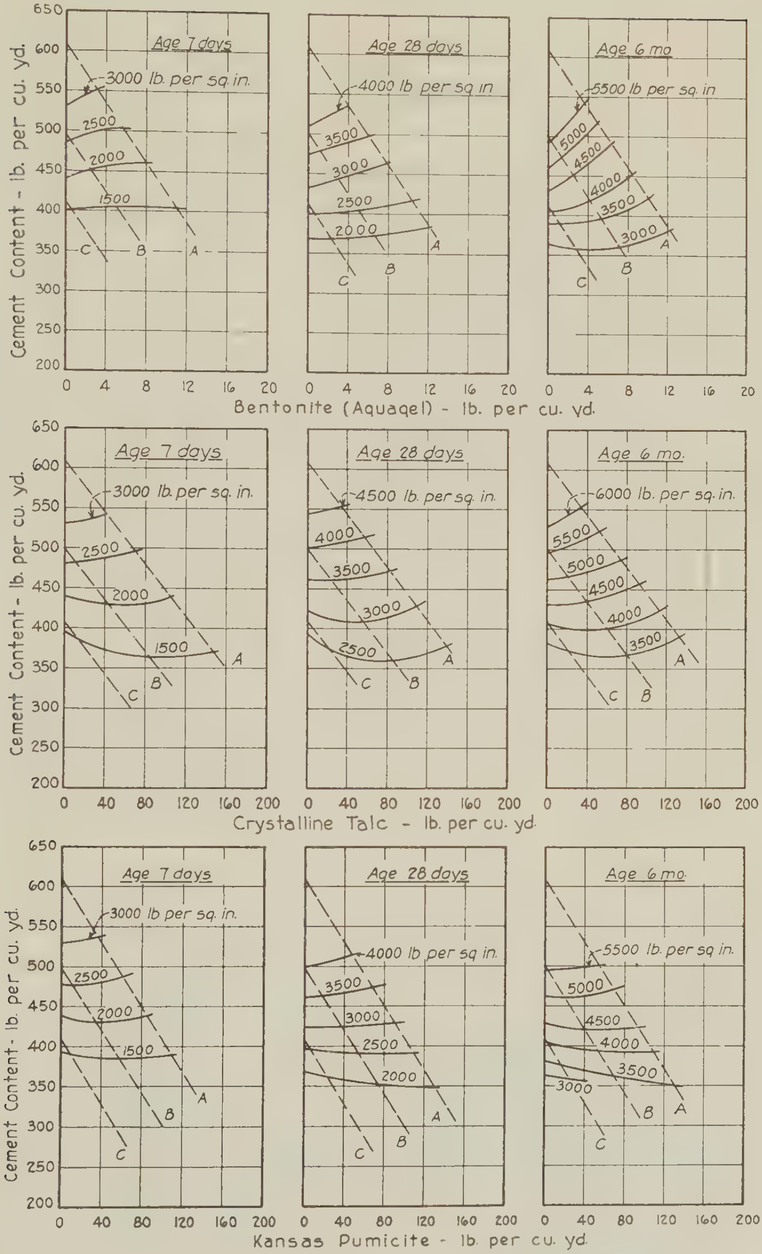
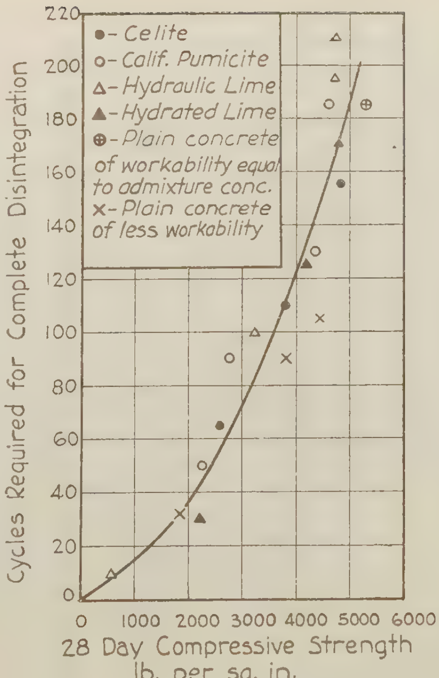


FIG. 5—EFFECT OF ADMIXTURES ON CEMENT REQUIREMENTS OF CONCRETE

Broken lines connect points of equal workability.



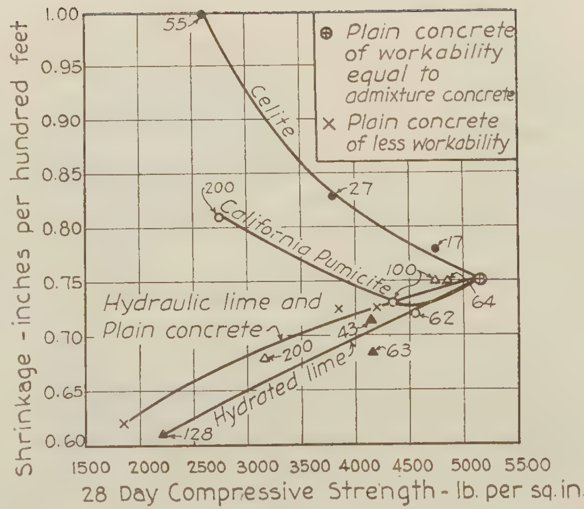


FIG. 6 (LEFT) —RELATION BETWEEN RESISTANCE TO DISINTEGRATION OF 6 BY 2-IN. DISCS BY FREEZING AND THAWING AND COMPRESSIVE STRENGTH OF 3 BY 6-IN. CYLINDERS MADE FROM THE SAME BATCHES

Standard curing conditions up to the age of 28 days when freezing and thawing began. One cycle required 24 hours. All concretes except those indicated by (x) had substantially equal workability.

FIG. 7—RELATION BETWEEN SHRINKAGE AND 28-DAY COMPRESSIVE STRENGTH FOR CONCRETES WITH AND WITHOUT ADMIXTURES

Numerals indicate quantity of admixture in lb. per cu. yd. Shrinkages are changes in length of concrete prisms cured moist for 7 days and for 6 months in air at 50 per cent relative humidity.

The Toronto Building By-Law*

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MEMBER AMERICAN CONCRETE INSTITUTE

1—HISTORY

Sixty years ago the first building by-law was passed by the Corporation of the City of Toronto at a time when the city had about one tenth its present population and the danger from conflagration was relatively greater than the dangers from defective construction. The first Inspector of Buildings was, in the words of the By-Law of 1890, to be "a competent, practical mechanic in the building trade." The technically trained plan examiner did not appear until much later.

Various amendments and revisions in the By-Law were made from time to time until in 1913 a very extensive revision was made by a committee composed of citizens representing all bodies closely concerned with the matter. Eleven years ago the chapters on structural design of the 1913 code were brought up to date by a committee of engineers and architects and that revision, with its subsequent amendments is the By-Law now in force.

2—PRESENT REVISION

Some two years ago, the Toronto Chapter of the Ontario Association of Architects and the Toronto Branch of the Engineering Institute of Canada appointed committees to consider a further revision. These acting jointly, built up a Main Committee by adding representatives of the Board of Trade, Industrial Commission, Building Managers, Association, General Contractors' Association and some other bodies' and obtained the hearty co-operation of the present Commissioner of Buildings. Sub-committee chairmen appointed by the main committee to deal with the various parts of the By-Law were empowered to appoint members to their sub-committees. The total personnel now engaged is about 65, many of whom are working on two or more sub-committees.

*Presented at the 30th Annual Convention American Concrete Institute, Toronto, Feb. 20-22, 1934
†Chapman & Oxley, Architects, Toronto, Canada.

The order of the By-Law aims to be that in which a designer having occasion to use it would have to take up the items. The chapters or "Parts" are as follows:

Part 1. Administration
- Section 1-A. Title and Scope
  - 1-B. Duties and Powers of the Building Department and the Public
  - 1-C. Methods of Enforcement
  - 1-D. Additional Permits Required

Part 2. Definitions

Part 3. Fire Limits and Height Restrictions
- Section 3-A. Fire Limits
  - Sub-section 3-A-1. Fire Limit A. etc.
- Section 3-B. Height Restrictions

Part 4. Occupancy Requirements

Part 5. Loads

Part 6. Materials and Stresses
- Section 6-A. Types of Construction
  - 6-B. Soil Loadings, Piles and Footings
  - 6-C. Masonry
  - 6-D. Reinforced Brick Masonry
  - 6-E. Concrete and Reinforced Concrete
  - 6-F. Haunched Beams
  - 6-G. Steel
  - 6-H. Cast Iron
  - 6-J. Timber

Part 7. Fire Resistive Standards

Part 8. Stair and Elevator Standards
- Section 8-A. Stairs
  - 8-B. Fire Escapes
  - 8-C. Ramps
  - 8-D. Elevator Enclosures

Part 9. Heating and Ventilating
- Section 9-A. Flues and Chimneys
  - 9-B. Boiler Rooms
  - 9-C. Heating Systems
  - 9-D. Ventilating Ducts

Part 10. Construction Regulations
- Section 10-A. Demolition of Buildings
  - 10-B. Regulations to be Observed in Construction
  - 10-C. Excavation and Shoring

Part 11. Encroachments

Part 12. Signs

The sub-committee on Concrete and Reinforced Concrete has been given three subjects to deal with, namely:

Concrete, Plain and Reinforced
Steel Beams Encased in Concrete
Reinforced Brick Masonry

So far, they have covered only the first of these, and even here the conclusions are subject to review and amendment after further consideration by the Main committee. What follows cannot yet be taken as the final form of the new By-Law, but the Main committee has permitted its presentation here due to the exigencies of the situation.

The order of main items in this chapter is as follows:

Definitions
Administrative Requirements
Material Specifications
Construction Requirements
Design Requirements

The important points under Administrative requirements are that at least 24 hours' notice be given of placing concrete to allow the Commissioner to have an inspector present, and that records shall be kept of the time and date of placing of each portion of the structure.

Provisions for the examination of the design plans, covered in the general administrative chapter of the By-Law, are not repeated here.

3—MATERIALS

Cement used shall be portland or high early strength portland, but special high early strength cements may be used subject to the requirements of the Commissioner.

The cement specifications for portland and high early strength portland follow the most recent specifications of the Canadian Engineering Standards Association except for the clause on Soundness which follows and are given at length in the By Law:

A pat of neat cement shall remain firm and hard, and show no signs of distortion, cracking, checking or disintegration in the steam test for soundness, which test shall consist of placing the cement, after storage in moist air for 24 hours, or until it is thoroughly set, in cold water which is raised to, and then maintained, at the boiling point for three hours.

Aggregate specifications also follow the Canadian Engineering Standards Association with the following provision for size of coarse aggregate:

The maximum size of the aggregate shall not exceed one-fifth of the narrowest dimension between forms of the member for which the concrete is to be used nor three-fourths of the minimum clear spacing between reinforcing bars. Where the above requirements do not govern, the maximum size of the aggregate shall not exceed 2 inches. By maximum size of aggregate is meant the clear space between the sides of the smallest square opening through which 95 per cent by weight of the material can be passed.

Coarse aggregates shall not exceed the maximum size required and shall not contain more than 10 per cent. passing the No. 4 sieve or 5 per cent passing the No. 8 sieve.

The following materials are prohibited for aggregate:

Pre-mixed or pit run aggregate
Broken brick or concrete, spalls and plums
Cinders—except in pre-cast cinder blocks.

Reinforcement shall comply with the Canadian Engineering Standards Association specifications.

Water shall be clean and free from oil, acid, alkali, organic matter or other deleterious substances.

4—STORAGE

Provision is made for the storage of materials to protect them from frost between Oct. 15 and April 15. At all seasons cement is to be fully protected from the weather or ground moisture, and in such a way as to permit ready identification of test lots. Aggregates shall not be in contact with the soil. Reinforcing shall not be in contact with the soil and shall be protected from damage or distortion.

5—CONCRETE QUALITY

The provisions for quality and strength of concrete are:

(a) The working stresses for the design of concrete structures shall be based upon the minimum ultimate 28-day strength of the concrete to be used in the structure in accordance with the values given in (b). All plans submitted for approval or used in the work shall show clearly the strength of concrete for which all parts of the structure were designed, the maximum size of aggregates permissible under the requirements of (c) and the consistency to be used, in terms of the slump. The strength of the concrete shall be fixed in terms of the water-cement ratio.

(b) Except as permitted in (d), the water-cement ratios used for different classes of concrete shall not exceed the values in the following table. The mixes shown in the table are the maximum volumes of aggregate that will be permitted for each volume of cement for the different water-cement ratios. The Commissioner may require that richer mixes shall be used if concrete of the proper workability is not being obtained within these limits.

TABLE 2

Minimum Permissible Mixtures Volume Portland Cement to Sum of Separate Volumes of Fine and Coarse Aggregates as Measured Dry

Water-Cement Ratio Imperial Gallons per 87½ lb. Sack of Cement	Plastic Concrete Slump up to 4 in.	Moderately Wet Concrete Slump 5-7 in.	Assumed Compressive Strength at 28 Days in p.s.i.
6.00	1:6.00	1:5.5	1500
5.50	1:5.25	1:4.75	2000
5.00	1:4.3	1:4.0	2500
4.50	1:3.75	1:3.25	3000

In interpreting this table, the surface water contained in the aggregate shall be included as part of the mixing water in computing the water-cement ratio. Where

the surface water is not determined accurately from tests, the following allowances shall be made for each cu. foot of fine and coarse aggregate in the batch:

	Condition of Aggregate	Imperial Gals. per Cubic Foot
Fine aggregate	Damp	1/3
	Moderately wet	1/2
	Wet	3/4
Coarse aggregate	Damp	No correction
	Wet	1/4

(c The proportions of aggregates to cement for concrete of any water-cement ratio shall be such as to produce proper workability, that is, concrete that will work readily into the corners and angles of the form and around the reinforcement without excessive puddling or spading and without permitting the materials to segregate or free water to collect on the surface.

(d Water-cement ratios greater than those given in Table 2, Article (b), may be used, provided that the proper water-cement ratios are established by tests made as provided in this article, and provided that the cement and aggregates are measured by weighing, using separate scales for the cement.

Where the water-cement ratios for the various strengths of concrete are to be established by test, these tests shall be made in advance of the beginning of operations using the materials proposed and consistencies suitable for the work and in accordance with the Method of Making Compression Tests of Concrete of the Canadian Engineering Standards Association including the provisions for curing in a moist room at 70° and testing wet. A curve representing the relation between the average 28-day strength of the concrete and water-cement ratio shall be established for a range of values including all of the strengths called for in the plans. The tests shall include at least four different water-cement ratios and at least four specimens for each water-cement ratio. The water-cement ratio used in the structure shall be that corresponding to a point on the *curve 20 per cent higher than* the minimum ultimate strength called for on the plans for all concrete designed to have a 28-day compressive strength *less than 2500 lb.* per sq. in. and *15 per cent* higher for all concrete designed to have a 28-day compressive strength *of 2500 lb.* per sq. in. or more. No substitution shall be made in the materials being used on the work without additional tests in accordance, herewith, to show the new water-cement ratios to be used.

6—MIXING

Requirements for job mixed concrete are for the most part conventional. On each job where mixing is done, a 12 qt. pail in good condition and a wooden box or other container built to hold exactly one cu. ft. of material which can be readily checked as to capacity must be available at all times for the use of the Commissioner or his representative to facilitate checking up the capacity of barrows or other containers at the job.

7—READY-MIXED CONCRETE

The only type of ready-mixed concrete provided for or allowed is that mixed in trucks in transit. The provisions are as follows:

Aggregates shall be stored in stockpiles and bins in such a way that the materials do not become mixed.

The cement and aggregates shall be measured by weighing on scales accurate to 0.4 per cent, and a separate scale shall be used for the cement. The cement scale shall be so constructed that it automatically indicates when the measuring hopper is completely emptied. The water shall be measured by weight or by volume. The device for the measurement of the water shall be readily adjustable and under all operating conditions shall be accurate to 0.5 per cent, or less of its maximum capacity.

Ready-mixed concrete shall be mixed in approved types of truck mixers. The maximum size of batch shall not exceed the rated capacity of the mixer as stated by the manufacturer and as stamped in metal at a prominent place on the mixer drum. The drum shall be watertight when closed. Each batch of concrete shall be mixed not less than 50 nor more than 150 revolutions of the drum at the rate of rotation specified by the manufacturer as mixing speed. If the concrete cannot be discharged immediately after mixing is complete, the concrete shall be agitated until discharged by rotating the drum at a lower speed as specified by the manufacturer for agitation. The drum shall be equipped with a discharge mechanism which permits of discharging the concrete without segregation. The truck mixer shall be equipped with a tank for carrying the mixing water; the water shall be measured at the proportioning plant. When the time of haul does not exceed 30 minutes, the water may be added directly to the mixer at the proportioning plant.

Concrete shall be delivered to the site of the work and discharge completed within a period of one hour after the introduction of the mixing water to the dry materials except that when the concrete materials are heated, this period shall be reduced to 30 minutes.

Concrete delivered in outdoor temperatures lower than 40° F. shall arrive at the work having a temperature not less than 60° F. nor greater than 100° F.

8—PLACING

The provisions covering placing follow the conventional form.

9—TESTS

In addition to tests on materials, cylinder tests on the concrete are required on all except very small and unimportant jobs. "Control" tests, made to ascertain whether the mixture is performing according to expectations, are called for at the rate of at least one specimen per 100 cu. yds., or three specimens of each strength of concrete in each day's operations. "Job" tests, made to ascertain the strength of the concrete cured on the job require an additional specimen for each 100 yards or two additional for each day's run of each strength. All specimens are 6 x 12 cylinders made according to the Canadian Engineering Standards Association specifications. At least one cylinder shall be tested at 7 days.

10—FORMS

Possibly the only item calling for remark under "Forms" is that no time limit is set for removal. The strength of the concrete is the criterion. However, the Commissioner has authority to prohibit removal of forms if he considers it necessary.

11—REINFORCEMENT

The requirements for reinforcement follow conventional lines except that no attempt is made to provide for "fireproofing" in the thickness of cover called for. These are based purely on questions of weather proofing or bond and fireproofing is left for handling in the Chapter on Fire Resistant Construction which gives hour ratings.

12—COLD WEATHER

Provisions for protection from cold must be on the job and ready for use at all times between Oct. 15 and Apr. 15 and shall be adequate for the following duties:

When the temperature of the surrounding atmosphere is 40° F. or lower, all aggregate and water shall be pre-heated, all reinforcing steel and forms shall be defrosted. Concrete when placed in the forms shall have a temperature of between 50° F. and 100° F. Suitable means shall be provided for maintaining a temperature of at least 50° F. for 4 days or as much longer as may be necessary to ensure the proper curing of the concrete. Canvas or other protective coverings shall be kept clear of all concrete to permit free circulation of air around all columns and over the tops of all slabs and beams.

Thermometers shall be provided and a temperature record kept of daily maximum and minimum temperatures during the curing period. A thermometer shall be placed at each of the four corners on the external side of the concrete between the concrete face and the protective covering. Thermometers at intermediate positions along the external face of the structure and about 75 ft. apart or as directed by the engineer, shall be provided. One thermometer shall be placed in the interior of the structure near the center. All thermometers shall be placed about midstory height but not over 7 feet above the floor.

On small or unimportant work, this requirement may be dispensed with if approved by the Commissioner in writing.

Dependence shall not be placed on salts, chemicals or other substances for the prevention of freezing.

13—CURING AND 14—JOINTS

Requirements for curing are conventional, as are also those for construction joints.

Expansion Joints—In all structures more than 200 ft. long, provision shall be made for expansion and contraction when considered necessary in the opinion of the Commissioner.

In all structures more than 150 ft. long, without a protective covering, and especially where there are marked changes in the section of the structure, expansion and contraction joints shall be provided.

The location and detail of all expansion and contraction joints shall be clearly shown on the plans. Joints shall be made entirely through the structure. Where double columns and beams are used, the double columns may be on a common footing.

Retaining walls, exclusive of basement walls, shall have expansion and contraction joints not farther apart than 60 ft.

15—DESIGN FORMULAE

The formulae given for design in general follow conventional practice with the exception of two-way slabs, but in all cases it is permissible for the designer to use other formulae according to the following clause:

Actual moment coefficients for the most severe loading conditions, determined by recognized engineering principles and based on relative rigidities, may be substituted for those specified in this Article.

One new item is as follows:

For the purpose of moment coefficients, the span lengths of beams may be considered equal when the maximum span length in a line of continuous beams of constant depth does not exceed the minimum span length by more than 25 per cent and when the live load is not more than four times the dead load.

This was based on the computation illustrated in Fig. 3 and later considered more fully.

Flat slab design follows the Canadian Engineering Standards Association requirements which in turn follow almost exactly the requirements of the 1928 Joint Committee Report.

16—WALLS AND PARTITIONS

Walls and partitions are covered rather more fully than has been the custom in previous codes as follows:

All reinforced concrete walls, whether bearing or non-bearing, shall be designed to sustain safely the vertical and horizontal loads which may come upon them. Exterior walls shall be designed to resist the wind pressure specified in Part 5. They shall be designed in accordance with the reinforced concrete column formulae except that no allowance shall be made for the reinforcing steel unless it is arranged and tied as specified in Article 6-E-27-E. (Column design)

All reinforced concrete walls shall be securely anchored to all intersecting walls, columns and floors. In non-bearing walls, provision shall be made for stresses arising from structural deflection of floors, beams or girders above and below the wall.

The minimum thickness of reinforced concrete bearing walls shall be not less than 1/25 of the unsupported height or length but shall be at least 6 in.

The thickness of exterior reinforced concrete non-bearing walls shall be not less than 6 inches. The distance between lateral supports shall not exceed 50 times the least dimension of the wall.

The thickness of interior reinforced concrete non-bearing walls shall not be less than 3 inches. The distance between lateral supports shall not exceed 50 times the least dimension of the wall.

Reinforced concrete walls shall be reinforced for temperatures and shrinkage stresses. In no case shall the bars be placed farther apart than 24 inches nor shall they be less than ⅜ inches in diameter.

The following shall be the minimum ratios of temperature and shrinkage reinforcement area to concrete area:

Exposed Walls—8 inches or more in thickness—		
Exterior face	—plain bars	.004
	—deformed bars	.0035
Interior face	—plain bars	.002
	—deformed bars	.0015
Exposed Walls—less than 8 inches in thickness—(bars may be in one layer)		
	—plain bars	.006
	—deformed bars	.005
Interior Walls—more than 6 inches in thickness—		
Each face	—plain bars	.001
	—deformed bars	.00075
Interior Walls—6 inches or less in thickness—in centre of wall—		
	—plain bars	.002
	—deformed bars	.00175
Walls below grade—face in contact with soil—		
	—plain bars	.001
	—deformed bars	.00075

The above percentages of reinforcement shall be placed approximately ⅓ vertically and ⅔ horizontally. Where the wall is designed as a slab or column, the main reinforcing steel shall not be less than is required by the above ratios and shall be considered as replacing the temperature and shrinkage reinforcement on that face.

17—FOOTINGS

Footings are treated practically as in the 1928 Joint Committee Report.

18—CAISSONS

Caissons, which is the name used locally for the members defined as "concrete foundation piers extending from the bottom of the columns of a structure to rock or hard pan," are covered as follows: (Local soil conditions permit excavation without lagging in many cases.)

The excavation for a caisson shall be maintained in its design form until concrete is placed. The center line of the required section shall be plumb.

The excavation for a caisson shall be free from standing water when concrete is placed.

When a caisson passes through a stratum of soft or flowing material, the least diameter of the caisson shall not be less than 1/7 of the depth of the soft or flowing stratum.

Concrete for caissons shall have a compressive strength at 28 days of not less than 2000 p.s.i.

A mortar cushion having the same proportions as the mortar in the concrete to be subsequently placed, and at least 6 inches thick shall be placed on the rock or the shale immediately before placing the concrete.

Concrete for caissons shall be placed in such a manner that it does not strike the sides, the reinforcing, lagging or other obstructions during its passage to the point of final deposit.

19—TWO WAY SLABS

Much study was given to rectangular slabs reinforced in two directions and supported on all four sides, perhaps more than the subject

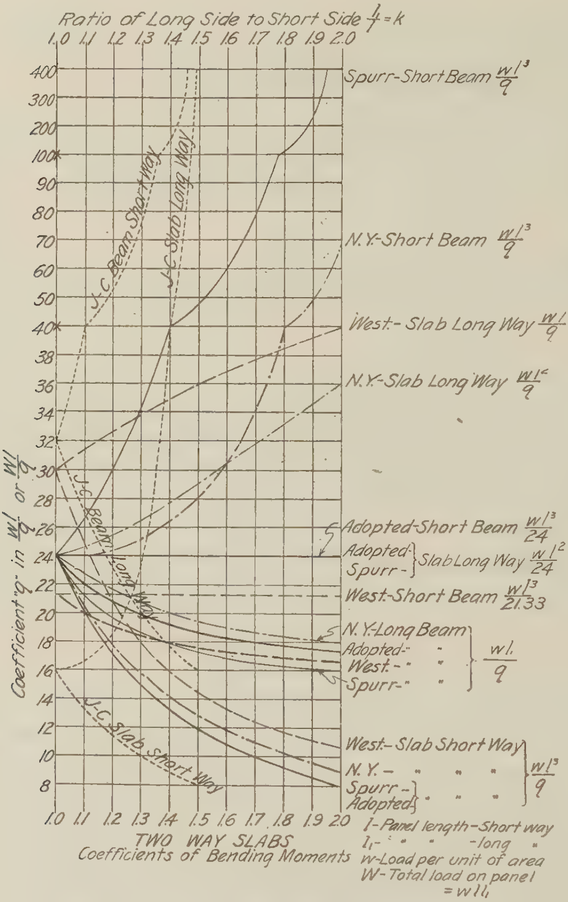


FIG. 1

warranted in relation to the amount of this type of construction used, but it was felt that most of the present codes did not give sufficient consideration to this subject and that the fixing of a more accurate and applicable method would be worth while.

The graphs shown in Fig. 1 give some idea of the variety of solutions which have been developed and used. Five sets of four curves each are shown here namely: Joint Committee 1928—which had many, many years of service and many acceptances for codes and specifications including the present Toronto By-Law; H. M. Westergaard as set forth in *Proceedings*, Amer. Concrete Inst., Vol. 22, 1926, probably the most thorough study of the subject available in English; Henry V. Spurr of New York, in a study made a few years ago; the proposed

New York building code now under consideration for adoption, and the formulae now proposed for adoption here.

In a few cases the curves coincide, but alas, in very few. The results of the studies of J. A. Wise by the elastic web method (*Proceedings*, Amer. Concrete Inst., Vol. 25, 1929) were also plotted for the slab moments and found to agree very closely with the curves shown for the proposed New York code for slab short way, and to be practically constant at $\frac{wl^2}{23.7}$ for the slab long way.

The aim of the committee in working with this problem was first, of course, to approximate the truth as closely as possible with the knowledge available, and secondly to express the results, if possible, in a form which would make them convenient for use by the designing engineer and the plan checker. The charting of the curves gave some idea of the lay of the land and incidentally showed pretty clearly that the old method need not be given any weight in arriving at a conclusion.

The first point to settle was the maximum positive moment in the slab of a square panel. The weight of evidence led to the adoption of $\frac{wl^2}{24}$ which agreed with New York, Spurr, and Wise and was 25 per cent greater than Westergaard's result. The curve for the moment in slab in short direction developed by Spurr was adopted.

Reasoning from Westergaard's studies as a basis two considerations led to this decision. The accepted formulae for Flat Slabs give a total coefficient of 0.09 for the sum of positive and negative bending moments as compared with 0.125 according to the regular principles of statics. This reduction, which amounts to 28 per cent is applied by Westergaard to the slab only, and the slab moment thus reduced is deducted from 0.125 to find the moment taken by the beams.

Is it not reasonable to assume that the whole structure, consisting of slab and beams, should be considered as analogous to the flat slab and that at least some portion of this reduction should be allotted to the beams?

The co-efficients proposed give a total of slightly over 0.125 for the sum of positive and negative moments and a little higher than Westergaard's total for a single panel, but somewhat under his totals for a panel in a continuous series.

The second consideration was the relation of negative to positive moments in the slab. Westergaard deduces a negative moment at

supporting beams as much as 50 per cent higher than the positive moment near the middle of the slab in some cases. From the point of view of practical construction this would be difficult to provide for as, if the slab were of balanced design for the positive moment, it would have to be thickened near the supports. This would be particularly awkward if tile fillers were used in the slab. It is, therefore, simpler to make the slab slightly overstrong for positive moments, so that it may be strong enough for negative moments without any thickening.

The moments adopted and charted (Fig. 1) are as follows:

Slab—short way........................ $\frac{wl_2\,(2k—1)}{24}$

Slab—long way........................ $\frac{wl^2}{24}$

Beam—short way........................ $\frac{wl^3}{24}$

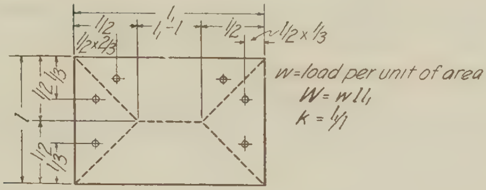
Beam—long way........................ $\frac{Wl_1}{16}\left(1—\frac{1}{3k^2}\right)$

These moments have the following relations to those developed by Westergaard:

Slab—short way........................25 to 34 per cent greater
Slab—long way........................25 to 42 per cent greater
Beam—short way........................11 per cent less
Beam—long way........................11 to 4 per cent less

It is admitted that this may give a slab slightly heavier than necessary but the difference when expressed in terms of slab thickness or amount of reinforcement is so slight as to be negligible. For the moment on slab in long direction the conclusion was reached that a constant moment regardless of the ratio of the sides of the panel should be used on the following reasoning: (Fig. 2) the distribution of load to supporting beams will be something of the nature indicated here due to equal deflections. The load from any point on a line drawn at 45 deg. from a corner will distribute in very nearly equal parts to the two beams forming that corner. Thus the load coming to the beam on the short side will be very nearly constant in relation to the length of the short beam regardless of the length of the long beam. The load and deflection on the slab tributary to the short beam will also be nearly constant in the same way, and starting from the square panel as a basis the curve representing the moment in the slab this way can be taken as a straight line.

Similar reasoning gives a constant co-efficient for the moment on the short beam itself and the load distribution indicated gives the moment adopted for the beam on the long side.



Moment on Short Beam

Reaction $= w(l/2)^2 \times l/2$ Moment arm $= l/3$

Moment $= \frac{wl^2}{8} \times l/3 = \frac{wl^2}{24}$

Moment on Long Beam

$$= \left\{w\frac{l_1 l}{2} \times \frac{l}{2} \times \frac{l_1}{4}\right\} - \left\{\left[w\left(\frac{l}{2}\right)^2 \times \frac{l}{2}\right]\frac{l}{6}\right\}$$

$$= \frac{wll_1^2}{16} - \frac{wl^3}{48} \quad = \frac{wl}{16}\left(l_1^2 - \frac{l^2}{3}\right)$$

$$= \frac{wll_1^2}{16}\left(1 - \frac{1}{3k^2}\right) \text{ or } \frac{Wl_1}{16}\left(1 - \frac{1}{3k^2}\right)$$

FIG. 2

It will be noted that Westergaard also finds a constant moment for the beam on short side at a slightly greater amount than proposed here, but that the moment for slab long way developed in his paper reduces by one-third from a square panel to one with a ratio of 2 to 1.

The effect of continuity in slab moments was considered and the following results tabulated:

RATIO OF MOMENTS IN CONTINUOUS SPAN TO SIMPLE SPAN

	Westergaard	Spurr	New York	Adopted
Pos. M. two span	0.80 to 0.57	0.85	0.80	0.80
Pos. M. multiple span	0.67 to 0.42	0.75	0.66⅔	0.66⅔
Neg. M. two span	1.20 to 1.035	0.85	1.00	1.00
Neg. M. 1st Int. support of multiple span	1.20 to 1.035	0.75	0.89	1.00
Neg. M. Interior supports	0.77 to 0.69	0.75	0.80	0.80

The ratios given under Westergaard are for the moment as shown by Westergaard for the case noted as compared with the moment adopted for the By-Law for the simple span.

For the moments in side beams in continuous spans it was assumed that the same ratios could safely be used as for beams in other situations.

While no attempt was made at a new mathematical analysis of the problem it was felt that the solution adopted was close enough to the truth to give safe results and that the formulae expressing this solution were not so fearsome as to frighten away designers from a very useful and in many cases economical type of construction.

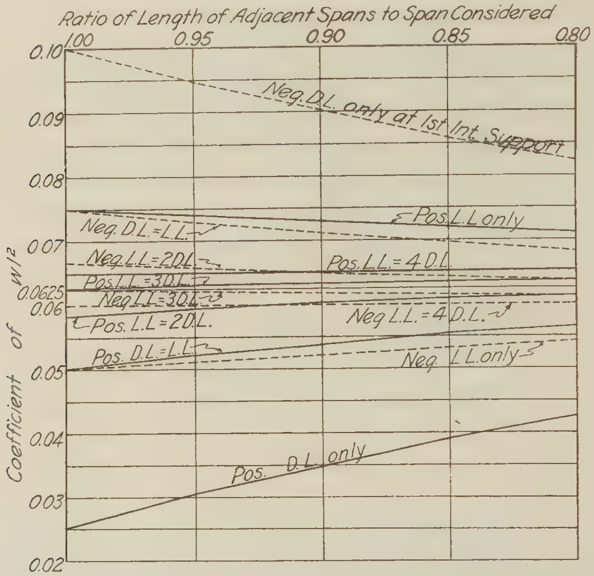


FIG. 3—BENDING MOMENTS IN CONTINUOUS BEAMS IN RELATION TO RELATIVE SPAN LENGTHS AND RATIO OF LIVE AND DEAD LOAD

Equal Spans

The clauses affected are the following:

Beams and slabs of equal spans freely supported or built to act integrally with beams, girders, or other slightly restraining supports and assumed to carry uniformly distributed loads shall be designed for the following moments at critical sections, provided that the supports are not reinforced to resist the moments due to such integral action.

Beams and slabs of one span:

Maximum positive moment near center,

M.................................. $\frac{wl^2}{8}$

Beams and slabs continuous for two spans only:

Maximum positive moment near center,

M.................................. $\frac{wl^2}{10}$

Negative moment over interior support,

M.................................. $\frac{wl^2}{8}$

Beams and slabs continuous for more than two spans:

Maximum positive moment near center and negative moment at support of intermediate spans,

M.................................. $\frac{wl^2}{12}$

Maximum positive moment near centers of end spans and negative moment at first interior support,

$$M \ldots\ldots \frac{wl^2}{10}$$

Negative moment at end supports for preceding cases in this article,

$$M \ldots\ldots \text{not less than } \frac{wl^2}{24}$$

Beams and slabs of equal spans built to act integrally with columns, walls or other restraining supports and assumed to carry uniformly distributed loads, shall be designed for the following moments at critical sections, provided that the supports are reinforced to resist the moment due to such integral action.

Intermediate spans:

Negative moment at interior supports except the first,

$$M \ldots\ldots \frac{wl^2}{12}$$

Maximum positive moment near centers of intermediate spans,

$$M \ldots\ldots \frac{wl^2}{16}$$

A study of the moments in beams of various relative span lengths and ratios of live load to dead load gave the results shown in Fig. 3.

As will be seen at a glance, negative moments are decreased in all cases by a shortening of the adjacent spans, except in the case of live load only, which, of course, cannot occur in actual construction.

Positive moments tend to increase with a shortening of adjacent spans, but in no case do they exceed the co-efficient of 0.0833 or $\frac{wl^2}{12}$. The particular case illustrated is for a line of three spans. Similar studies for live span combinations produced similar results. As no allowance whatever is made for fixity at supports, and as in practice, there will always be a certain amount of fixity, it seemed reasonable to adopt the limits given.

For such discussion of this paper as may develop, readers are referred to the JOURNAL *for October, 1934 (Proceedings, Vol. 31). Discussions should be available to the Secretary by August 1, 1934.*

Experimental Study of Stresses at a crack in a Compression Member*

BY S. C. HOLLISTER†
MEMBER AMERICAN CONCRETE INSTITUTE

This paper reports the results of a series of photo-elastic tests made to study stresses at a crack occurring in a compression region. They were undertaken upon the suggestion of Dr. H. M. Westergaard.‡ The investigation was carried out by the writer in the photo-elastic laboratory of Purdue University, School of Civil Engineering.

FORM OF SPECIMEN

All specimens used in this investigation were made of selected celluloid and were ¼ in. thick and 1½ in. wide. The length of the piece each way from the crack was 3½ in. The ends were cemented to steel blocks 1 in. high by 2 in. wide by ⅝ in. thick. Loading was imparted to the steel blocks through ¼-in. rollers.

It was necessary to stay the specimen in the direction of its thickness to prevent buckling. This was accomplished by providing two guides between which the specimen passed.

In preparing these specimens which had surfaces in bearing, the contact surfaces were formed by grinding the pieces against each other using a fine abrasive paste.

The total load applied to a specimen did not exceed 200 lb.

TESTING PROCEDURE

The specimen was loaded by means of a lever system, and was placed in a field of circularly polarized light. The resulting image was projected upon a screen. As the load was applied, color bands appeared upon the screen. Lines representing the path of fringes of the same color, called "isochromatics," represent loci of points of equal difference of the two principal stresses.

At the net section where a horizontal crack occurs in a compression piece, the lines of principal stress are vertical and horizontal, the applied compression being assumed to be applied vertically. While

*Presented at the 30th Annual Convention American Concrete Institute, Toronto, Feb. 20-22, 1934 .
†Professor of Structural Engineering, Purdue University, Lafayette, Ind.
‡Doctor Westergaard's paper, "Stresses at a Crack, Size of the Crack and the Bending of Reinforced Concrete," also presented at the 30th Annual Convention, was published, JOURNAL of the Amer. Concrete Inst., Nov.-Dec., 1933, *Proceedings* Vol. 30, p. 93.—EDITOR

the specimen was under load, a tension bar of material from the same sheet from which the specimen was made was placed in the same path of light with the axis of tension coincident with the axis of principal compression in the specimen. The tension bar was then strained until the color image at a given point on the net section was reduced to a black shadow. The pull on the tension bar was noted, from which the tensile unit stress in the tension bar was computed. This tensile unit stress represented quantitatively the value of the principal stress-difference at the point on the net section of the specimen where the shadow appeared.

The operation was repeated for successive points along the net section of the specimen.

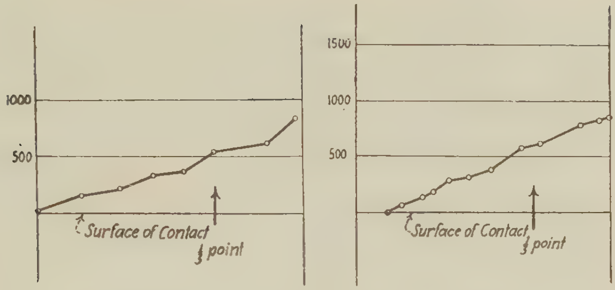
DATA OBTAINED FROM THE TESTS

The separate values of the two principal stresses were not readily obtainable by the method employed in the tests. These values are therefore not reported here. The data presented represent the *difference* in amounts of principal stress at successive points along the net section expressed in pounds per square inch. At any unloaded boundary of the specimen the principal stress normal to the boundary must of course be zero at the boundary. At such points, therefore, the stress parallel to the boundary has the same value as the stress-difference at the boundary.

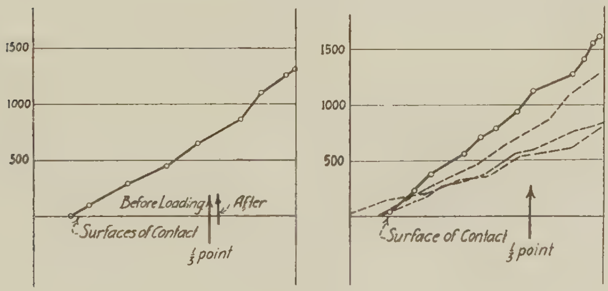
The first series of tests was made upon a specimen having a surface of contact over the full width. These were bearing surfaces and were not cemented together. The specimen was first subjected to a central loading and tested for uniformity of stress distribution, to be certain that the steel loading blocks were sufficiently rigid not to introduce end effects in the specimen. The load was next placed at the third point of the width of the specimen. The resulting measurements of stress-difference are shown on Fig. 1*a*.

It should here be noted that observational variations of a small order are possible in the method of testing employed, because at times the shadow formed by the superposition of the tension bar image upon the image of the specimen is of appreciable width. Although the dynamometer used on the tension bar was sensitive to the nearest 14 p.s.i., some error is introduced in the separate observations by error in judgment in locating the core of the shadow brush.

As shown in Fig. 1*a*, the stress-difference variation on the cross-section appears to be a straight line. The same appears to be true when the loading was increased, as shown in Fig. 1*b*. In this figure it is noted that the point of zero stress-difference has moved away from



(a) FIG. 1 (b)

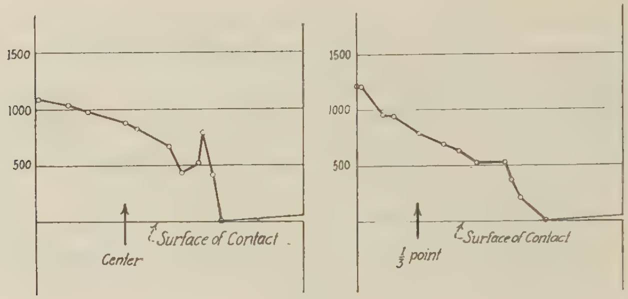


(a) FIG. 2 (b)

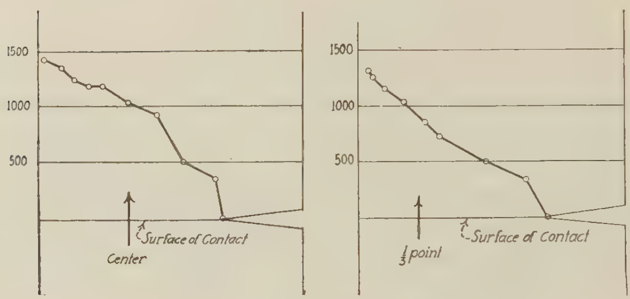
the left edge of the specimen. This point was investigated at a higher loading shown on Fig. 2*a*. It was found that the movement of the zero point was due to a material lateral deflection of the specimen, due to the eccentric strain. The line of loading shifted, as is shown on the figure.

A still further advanced bulletin is shown on Fig. 2*b*. On the same figure are reproduced the results shown in the three preceding figures. This clearly shows the shift of the point of inflection as lateral deflection of the eccentrically loaded specimen increases.

Remaining figures of this paper show in each case the axis upon which the load line was initially placed. The actual amount of shift of the load line is not represented, but its amount can be estimated approximately by reference to Fig. 2*a*.



(a) FIG. 3 (b)



(a) FIG. 4 (b)

The next specimen tested was similar to the last, except that a bevel of one of the two pieces in contact was formed to extend to a point a half-inch from the edge, or a distance equal to one-third of the width of the specimen. The specimen was then loaded at the center of the remaining surface of contact. The stress-difference values obtained from this loading are plotted on Fig. 3*a*. The specimen was then shifted to the third-point of the surface of contact, the resulting stress-difference measurements for which are shown on Fig. 3*b*.

A specimen was then prepared with the bevel formed on each side of the surface of contact, as shown on Fig. 4*a*. This figure shows the stress-difference measurements when the loading was applied at the center of the surface of contact. The loading was then shifted to the third-point, the results for which are plotted on Fig. 4*b*.

A specimen was then formed of one piece with a transverse cut extending through the specimen for a third of its width. Loading was then applied at various points located with respect to the remaining

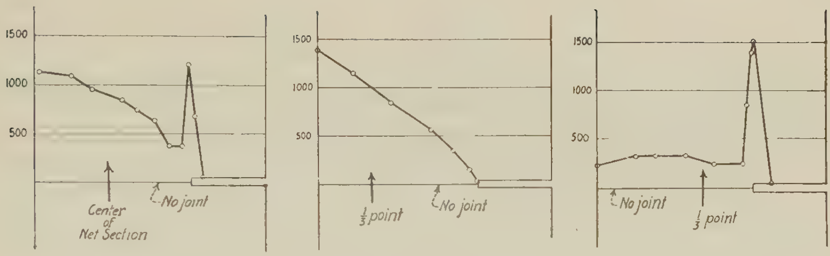


FIG. 5
(a) (b) (c)

net section. The stress-difference values obtained with a loading at the center of the net section is shown on Fig. 5*a*. The load was then shifted to the third-point farthest from the notch, the results obtained being shown on Fig. 5*b*. Finally, the load was shifted to the third-point nearest the notch, the resulting stress-differences for which condition are shown on Fig. 5*c*.

DISCUSSION OF THE RESULTS

The writer greatly regrets that with the apparatus and time available the separation of the two principal stresses could not be made. He feels, however, that there is sufficient significant information in the measured stress-differences here reported to make them of some interest. This paper is a progress report; study is being continued.

The first significant feature of the results is the absence of a high stress concentration typical of tension members having notches or cracks, unless the resultant of the pressure force lies on the side of the net section toward the crack, as shown in Fig. 5*c*.

The distribution of stress-difference appears to vary with the position of the resultant compression, with respect to the net section, as well as with the form of the notch or crack.

For such discussion of this paper as may develop, readers are referred to the JOURNAL *for October, 1934 (Proceedings, Vol. 31). Discussions should be available to the Secretary by August 1, 1934.*

Proposed Standard Specification for the Design and Construction of Reinforced Concrete Chimneys

Report of Committee 505—Reinforced Concrete Chimneys

E. A. DOCKSTADER, AUTHOR-CHAIRMAN

Committee 505 has submitted for tentative adoption by the Institute a Specification for the Design and Construction of Reinforced Concrete Chimneys. In conformity with provisions of the By-Laws (Article 5—Standards) as amended in 1933, the specification will not now be published in the Journal. *Copies of the complete text and the numerous diagrams will shortly be available. Any member of the Institute may have one copy on request addressed to the Secretary of the Institute prior to Feb. 1, 1935. Additional copies will be supplied to members at 75 cents each—to non-members, $1.25 per copy. Following is the report in outline, prepared by Mr. Dockstader and approved by critic members of the Committee: E. R. Maurer, K. B. Foster, William Jassoy. These critic members have also approved the report as a whole.*—Secretary.

The specification sets forth recommended loadings, including provision for both wind and earthquake, for the design of reinforced concrete chimneys and recommended methods for determining the stresses in the concrete and reinforcement resulting from these loadings. The method of analysis includes determination of the stresses at horizontal cross sections where flue or other openings occur as well as at sections where the cross section is an annular ring. Charts containing curves to aid in the rapid solution of the specified formulae are included. While the method of analysis applies primarily to chimneys, it can be used for other hollow circular cross sections, with or without openings, where the shell thickness is small in proportion to the diameter.

Recommended formulae are given for determining the temperature gradient through the concrete resulting from the difference in temperature of the gases inside the chimney and the surrounding atmosphere, together with methods for determining the stresses in the concrete and reinforcement both vertically and circumferentially due to the temperature gradient through the concrete.

Formulae for combining the stresses due to dead load, wind and earthquake with the stresses due to temperature are included in the specification, together with recommended allowable stresses in the concrete and reinforcement for the various stress combinations.

The specification includes provisions concerning the mixing and placing of the concrete, but as the requirements of this phase of the construction of concrete chimneys are comparable with those involved in other high grade concrete construction and as these requirements have been and will continue to be the subject of full discussion in the meetings and literature of the Institute, the provisions of the specification in this respect are of a general nature. A recommendation regarding the curing of the concrete is included as conditions in this respect for chimneys are somewhat unusual.

The specification also includes recommended practice for linings for concrete chimneys, where required, for lightning protection, access ladders and other chimney accessories.

An appendix to the specification covers the derivation of all of the formulae used in the specification, in which are set forth the assumptions on which the formulae are based.

The Committee believes that some features of the specification, particularly the treatment of stresses at sections where openings in the chimney wall occur, of stresses due to temperature and the methods set forth for combining temperature stresses with other stresses will be found more complete than have heretofore been available in American engineering literature and trusts that they will prove of interest and value to the membership of the Institute and to the engineering profession.

In preparing the specification the Committee has proceeded on the basis that it was advisable to include a definite method for determining the more important stresses in a reinforced concrete chimney, together with definite limits for these stresses even though the accuracy of the method of analysis was subject to discussion. The Committee will welcome discussion and criticism by members of the Institute of the specifications as submitted for tentative adoption.

Properties of Mortars and Concretes Containing High-Silica Cements*

BY RAYMOND E. DAVIS, R. W. CARLSON, J. W. KELLY, AND G. E. TROXELL†

MEMBERS AMERICAN CONCRETE INSTITUTE

SUMMARY

THIS PAPER defines high-silica cements as (a) low-lime portland cements containing a high percentage of dicalcium silicate, (b) puzzolan-portland cements produced by blending an active siliceous material (puzzolan) with a portland cement, (c) puzzolan-lime cements, produced by blending an active siliceous material with lime, and (d) sand cements, produced by intergrinding a relatively inactive siliceous material with portland cement.

Among the materials which are considered as puzzolanic in character are (a) those of volcanic origin such as pumicite, tuff, trass, and pozzuolana, (b) those of diatomaceous origin, such as diatomaceous earth and diatomaceous shale, and (c) those that are artificially produced, such as blast-furnace slag and burned clay.

The use of puzzolan-lime and puzzolan-portland cements is extensive throughout Europe and in Japan. The original puzzolan-lime cements date back to the time of the early Romans, and the use of puzzolan-portland cements extends over almost a century.

In the United States until recently the use of puzzolan cements has been restricted to slag cements and to the so-called "tufa" cement which was employed in the construction of the Los Angeles Aqueduct. Recently there has been an increased interest in cements of this type, and cements are now being produced and used in which the puzzolanic materials are pumicite, tuff, and diatomaceous shale. Sand cements were extensively used at the beginning of the century in the construction of government works, principally dams.

*Presented at the 30th Annual Convention, American Concrete Institute, Toronto, Feb. 20-22, 1934.

†The Authors, all of the University of California are respectively: Professor of Civil Engineering, Research Engineer, Engineering Materials Laboratory, Research Engineer, Engineering Materials Laboratory, and Associate Professor of Civil Engineering.

From the evidence available it appears that in general puzzolan-portland cements have the advantages over normal portland cements of greater resistance to the action of acids and alkalis, lower heat of hydration, greater freedom from the leaching of soluble compounds, and a greater workability for concretes.

In general the disadvantages appear to be the necessity for a longer period of moist curing to develop optimum strength, and greater shrinkage in concrete subject to drying conditions.

The French investigations of Vicat, Le Chatelier, and Feret, extending over a century, all point to the primary cause of disintegration of concrete in sea water as being due to the action of magnesium and sodium sulphates on lime liberated by the cement, and that favorable results are obtained by the addition to portland cements of properly chosen puzzolanic materials. The investigations of Michaelis in Germany, Poulsen in Denmark, Manonilov in Russia, and Hiroi in Japan point to the same conclusions, and several foreign countries specify the use of cements of this type for concrete construction in sea water.

In the United States, investigations carried on by Bates, Phillips and Wig (reported in 1912), by Hughes and Levens (reported in 1930) and by the Los Angeles County Flood Control District, all tend to confirm the observations of foreign investigators. In 1933, Bates stated that "the puzzolanic cements have the outstanding properties of low early strength, but very steady gain in strength, and an excellent resistance to aggressive waters after they have acquired a certain part of their ultimate strength."

The results of investigations of high-silica cements, which have been in progress in the Engineering Materials Laboratory of the University of California since 1931, indicate that cements of this type have possibilities for a variety of concrete structures, even though many of the test conditions were designed particularly to develop the imperfections of the puzzolanic cements included in the program. Such evidence as has been collected indicates that puzzolan-portland cements manufactured from proper materials and under proper conditions are consistently sound, are highly resistant to the action of sulphate waters, and are of low heat of hydration. Concretes produced by using these cements exhibit satisfactory strengths as compared with corresponding portland cements, not only at the later ages but at the early ages as well.

There appears the additional interesting possibility of producing puzzolan-portland cements, where raw materials are readily available,

at a lower cost than portland cements having the same general characteristics.

At the same time it is recognized that puzzolanic materials differ very widely in their chemical and physical characteristics and that, as a consequence, among puzzolan-portland cement concretes there may be corresponding wide variations in their chemical and physical properties. Past experience in the use of cements of this type indicates that, just as with portland cements, factors which may be favorable for one combination of conditions may be unfavorable for another.

The recent interest that has developed in this country among cement manufacturers and users in the possibilities of puzzolan-portland cements, and the fact that a number of manufacturers are either producing or preparing to produce puzzolan-portland cements, indicate the need for more comprehensive information concerning the effect of the composition of these cements upon strength, heat of hydration, resistance to aggressive waters and the action of weather, volume changes, etc. of concretes subject to the variety of conditions encountered in construction. To meet this need there has recently been inaugurated a more comprehensive series of investigations in the Engineering Materials Laboratory of the University of California, involving the manufacture and testing of 60 puzzolan-portland cements varying from one another in the chemical composition of the portland-cement clinker and in the chemical composition, physical character, and proportions of puzzolanic material.

Among the siliceous materials included in the investigation are (a) granite such as was used in the sand cement for the Arrowrock Dam, (b) volcanic tuff such as was used in the so-called "tufa" cement employed in the construction of the Los Angeles Aqueduct, (c) Fresno pumicite such as has been used in certain Los Angeles County Flood Control dams and in numerous other structures in California, (d) diatomaceous earth, (e) diatomaceous shale such as is used in the high-silica cement now being employed in the construction of piers for the Golden Gate Bridge, (f) ordinary siliceous clay, (g) quartzite, and a number of other similar materials. In the preparation of these cements, the siliceous materials are employed both in the calcined and the uncalcined state in percentages varying from 10 to 45, and all are interground to two degrees of fineness with portland-cement clinker.

The investigations, which are being carried out in cooperation with several other agencies, include tests to determine strength, resistance to the action of sulphate waters, resistance to the action of weather,

heat of hydration, volume changes, etc., under a wide variety of curing conditions.

The program is designed to develop (a) a reliable method of test to determine the suitability of various types of puzzolan, (b) the optimum percentages of the various types of puzzolan to be employed, (c) the most satisfactory chemical composition of portland-cement clinker to be interground with the puzzolan, and (d) the optimum degree of fineness, considering not only the physical and chemical properties of the resulting concretes, but the cost as well. The ultimate aim of the investigation is to provide information which may be used as a guide in the preparation of specifications for cements of this type and for their use in concrete construction.

INTRODUCTION

Portland cements blended with various siliceous materials have been used long and extensively in Europe for marine structures, and to a limited extent in the United States, chiefly for dams and where the primary consideration was that of cost. Interest in their use has been stimulated recently by the desire (a) in mass concretes, to limit the temperature rise during the hardening period and thereby to reduce the subsequent contraction upon cooling, and (b) to secure concretes which are resistant to corrosive waters or soils.

The purpose of this paper is to describe briefly the past use and the properties of high-silica cements, and to present in greater detail the results of investigations either completed or now under way in the Engineering Materials Laboratory of the University of California. The accumulated wealth of data on the subject is scattered, is incomplete in many respects, and is often conflicting, probably because of differences in either the silica, the cement, the method of use, or the climate and the curing conditions.

TYPES OF HIGH-SILICA CEMENTS

The term "high-silica" when applied to cement is the general term covering all of the following types and perhaps others.

1. Low-lime portland cements burned in the normal manner from raw materials containing a high percentage of silica (SiO_2), without subsequent admixture other than a retarder, and thus containing a high percentage of dicalcium silicate (C_2S).

2. Puzzolan-portland cements made by blending an active siliceous material (puzzolan) with a portland cement, either by intergrinding or by simply adding the puzzolan to the cement (see definition of puzzolan, page 373). The active silica of the puzzolan combines with the lime set free during the hydration of the portland cement, forming stable calcium hydro-silicates which have cementing value.

3. Puzzolan-lime cements made by blending a puzzolan with lime. This classification includes *slag cements* made by intergrinding granulated blast-furnace slag with lime. Puzzolan-lime cements are often used in combination with portland cements.

4. Sand cements made by intergrinding a relatively inactive siliceous material (such as natural or crushed sand) with portland cement.

This paper is concerned chiefly with puzzolan-portland cements.

Definitions

1. Puzzolan.—The general term for a siliceous material which in itself has no cementing value but which exhibits cementitious properties when mixed with hydrated lime.

2. Volcanic Ash.—The general term for material expelled from a volcano in a finely divided state, which material may later become consolidated.

3. Pozzuolana (pronounced "potswolana").—A volcanic ash or siliceous rock of volcanic origin, found chiefly near Pozzuoli, Italy, but also in southern France and neighboring regions. Herein the term refers to these particular deposits only.

4. Pumicite.—A volcanic ash chemically similar to pumice, occurring finely divided in nature (see "pumice").

5. Tuff.—A consolidated volcanic ash. See "pumicite," "tufa," and "trass."

6. Tufa.—A porous rock formed as a deposit from springs or streams. Often this term has been used loosely for "tuff," but geologists and petrographers limit its meaning as here defined.

7. Trass.—A light-colored volcanic tuff resembling pozzuolana, found chiefly in Germany. Eckel defines trass as an ancient volcanic mud.

8. Santorin Earth or *Santorin.*—A fine, light gray, siliceous material of volcanic origin, found on the island of Santorin, near Greece.

9. Infusorial Earth.—Siliceous remains of infusoria (minute sea animals), loosely classified with diatomaceous earth.

10. Diatomaceous Earth or *Kieselguhr.*—Siliceous remains of diatoms (minute sea plants). See "tripoli."

11. Diatomaceous shale.—Consolidated diatomaceous earth.

12. Tripoli.—A siliceous earth (tripoli powder) or friable mass (tripoli stone) originally found at Tripoli, Africa, and consisting chiefly of remains of diatoms.

13. Blast-furnace Slag.—The non-metallic product, consisting essentially of silicates and alumino-silicates of lime, which is developed simultaneously with iron in the blast furnace.

14. Lava.—Fluid rock such as issues from a volcano, or such rock solidified.

15. Pumice or *Pumice Stone.*—A hardened volcanic glass froth, usually occurring above lava. See "pumicite" and "scoria."

16. Scoria.—A hardened volcanic glass froth similar to pumice but more glassy and having larger cells.

17. Arenes.—In France, feebly hydraulic sands and residual material derived from decay of igneous rocks; much used in early period of development; practically worthless today.

18. Sand.—In connection with blended cements, a granular siliceous material of low activity, either natural or crushed.

19. Bentonite.—A colloidal clay capable of forming gel with large percentages of water.

20. Burned Clay.—Best known of the minor puzzolanic products; usually employed in the form of powdered brick or tile.

Puzzolans

The chief puzzolans which have been used in concrete construction are classified as follows:

1. Of volcanic origin—Not consolidated (pumicite, santorin) and consolidated (pozzuolana, tuff, trass).
2. Of diatomaceous origin—Not consolidated (diatomaceous earth, kieselguhr, tripoli powder) and consolidated (diatomaceous shale, tripoli rock).
3. Artificial (blast-furnace slag, burned clay).
4. Deposits from thermal springs (tufa).

Natural puzzolans occur in many parts of the United States, chiefly in the West. The most abundant deposits are in California and Kansas. Recent developments in the use of puzzolan-portland cements have been concerned chiefly with pumicite, tuff, diatomaceous earth, and diatomaceous shale.

Except in unusual cases, sand is active only to a slight extent, and it is not at present considered as a puzzolan.

Puzzolans are composed largely of silica (SiO_2) and alumina (Al_2O_3), with some iron oxide (Fe_2O_3). The lime (CaO) content is low, except in blast-furnace slags. Following is given the analysis (ignited basis) of an average European puzzolan, as published by Eckel (3),* also the approximate range for the United States:

	Average of 31 European Puzzolans	Approximate Range in U. S.
SiO_2	55	45—90
Al_2O_3	18	2—20
Fe_2O_3	12	1—17
CaO	6	0—10
MgO	2	0—7
Alkalis (K_2O, Na_2O)	7	0—10

Slag cements usually (not necessarily) contain about 1 per cent of sulphides Opinions differ as to whether these sulphides oxidize on exposure to air, and it is sometimes considered desirable to use the product only underground or in large masses with relatively small surface area exposed.

The most desirable property of a puzzolanic material is apparently a high content of "active" silica. The activity is estimated indirectly either by determining the degree of solubility in acid or alkaline solutions, or by testing the strength of a mortar (with or without sand) containing fixed proportions of hydrated lime and the puzzolan. It may be determined directly by long-time tests for strength of mortar or concrete containing either lime and the puzzolan or portland cement and the puzzolan.

Most puzzolanic materials occur finely divided, or are readily reduced to fine particles by grinding. As in the case of portland cement, high fineness is apparently necessary for an appreciable degree of activity. The specific gravity of puzzolans ranges from 2.3 to 2.8, whereas that of portland cements is about 3.1; thus puzzolan-portland cements are of lighter unit weight than are portlands.

HISTORICAL NOTES

The early Romans used a puzzolan-lime cement composed of volcanic ash ("pozzuolana") and lime for many structures such as the retaining walls along the Tiber River and the aqueducts which supplied Rome with water, and for many buildings. The Roman invaders of

*References are to corresponding numbers in bibliography.

Germany used trass and lime in the construction of an aqueduct from Eifel to Cologne. Parts of these structures still exist. These or similar types of puzzolan-lime cements were the only ones used in construction until less than two centuries ago, when natural cements were developed.

About the middle of the 18th century, Smeaton in England experimented with lime in combination with trass, pumice, cinders, brick dust, etc. In the construction of the Eddystone lighthouse, he used a mortar made of equal parts of lime and a puzzolan (either trass or pozzuolana).

At the beginning of the 19th century, French investigators took the lead in the development of hydraulic cements. The French materials used as puzzolans up to that time were apparently not the equal of the Italian pozzuolana and the German trass, and a number of failures had occurred. Vicat's work over forty years was climaxed by his prize-winning paper in 1853, in which he stated that the primary cause of disintegration of concrete in sea water is the action of magnesium sulphate on the lime in excess of the quantity capable of being neutralized by the carbonic acid of the mortar (4). Le Chatelier, as a result of his own experiments and Vicat's work, proposed that the calcium hydrate be reduced or eliminated by the addition of a certain quantity of pozzuolana to the cement. Near the end of the 19th century, Feret concluded that favorable results could be obtained by the addition to portland cements of properly chosen puzzolanic materials. Tests of mortar in sea water have been carried on continuously since 1852, and have confirmed the value of puzzolanic additions to portland cements. Construction practice in France has largely conformed to these findings. The present French tentative specification for cements includes hydraulic limes, grappier cements, slag cements, and blended cements of various types (5).

In Germany, Michaelis (1882) stated that the excess calcium hydrate in gaged portland cement has a detrimental effect, and that trass or other puzzolans combine with this calcium hydrate to form calcium hydro-silicate, which is an effective cementing material. Michaelis' position was substantiated by extensive tests conducted by official government agencies, and trass-portland cements were widely used in harbor work. Later tests (1902-1909)were made by Burchartz, using trass as a replacement rather than an addition. As might be expected, the results were not as favorable as those of Michaelis (4). However, today the use of puzzolan-portland (trass or slag) cements in German marine work is general.

In England, puzzolans have not been used as generally as on the Continent, probably because supplies of natural materials were not available. However, extensive researches on artificial puzzolans have been conducted by the Department of Scientific and Industrial Research, including such materials as blast-furnace slag, powdered brick and tile, burned clay, cinders, and spent oil shale; and the value of these materials is recognized (6).

In Spain, numerous examples of resistant concrete in port construction are cited by de Castro (4). In 1915, a low-lime portland cement was manufactured for the construction of a large dam in an isolated location (7).

The early use of puzzolan-lime cements in Italy has been mentioned. Near the end of the 19th century, Italian harbor engineers began to use portland cement. Luiggi states that since 1888 it has been common practice to add pozzuolana to portland-cement mortars and concretes, and that the results have been uniformly good (8, discussion).

In Denmark, Poulsen (1912) found that, for the conditions of his tests, diatomaceous earth was superior to trass as the puzzolanic component of cements for sea-water construction (8, discussion).

In Russia, Manonilov (1923) has summarized the experience of several countries with regard to the deterioration of portland cement in water somewhat as follows: "the simplest and cheapest method of preventing the decay of portland cement in water is that of introducing puzzolanic 'aggregate' into the concrete, the active silica being the main component of the above aggregate." (9, discussion). Numerous Russian port structures have been built using straight portland cement, also a number of structures in which Roman pozzuolana and Teil lime (containing some silica) were employed with portland cement. Based upon observation over several decades, the official standards now require puzzolanic substances to be used in connection with portland cement for port construction.

Japan has an abundance of volcanic puzzolans, and has been active in research and construction employing these materials. A puzzolanic cement was used in the construction of the Nagasaki Dry Dock, with good results (8, discussion). Hiroi, after tests extending over more than 20 years, concludes that there are considerable differences between puzzolanic materials, and that these differences may best be evaluated by physical tests (4). He states that the action of volcanic ash when used in a portland-cement mortar appears to be both mechanical and chemical. Mechanically the finely divided ash increases the density of the mix, and chemically it combines with the free lime of the cement. The active silica in an ash is important, but the total silica should also be taken into consideration. Hiroi cites the satisfactory resistance to sea-water exposure for 20 years of concrete blocks containing volcanic ash as part of the cement.

In Canada, Thorvaldson has studied the relative resistance of various cements to the action of sulphate waters. It was found that the presence of silica gel, the probable active ingredient in puzzolans, renders a portland cement very resistant to solutions of sodium sulphate (10). In tests of the effect of steam curing, it was found that calcium hydroxide formed in the early stages of cement hydration apparently combined with silica in the aggregate of the mortar.

United States.—In the United States, slag cements were used before the beginning of the 20th century. In 1901 there were 6 mills manufacturing slag cement, although the number is less today. Sand cements were used by the U. S. Army engineers for locks and dams on the Mississippi River; the 1901 Report of the Board of Engineers included specifications for slag cement and silica or sand cement (1). According to these specifications the slag cement was to be made by grinding granulated blast-furnace slag with slaked lime, without calcination. It was stated in the report that slag cement never becomes extremely hard like portland, but is tougher or less brittle than portland; it is "well adapted for use in sea water and generally in all positions where constantly exposed to moisture."

The use of sand cement was considered by the U. S. engineers to be a question of cost and expediency; also, on account of the extreme fineness resulting from the grinding of sand with portland cement, the mortar was considered to be more dense.

The outstanding example of the use of puzzolan-portland cement in the United States is the Los Angeles Aqueduct (1910-1912), in which more than 600,000 bbl. of tuff-portland cement were used (8). The tuff (called "tufa" by the builders) was a volcanic ash resembling German trass. Opinions differ as to whether the favorable test results obtained with the blended cement were due to activity of the silica in the tuff

or to the fineness of grinding (90 per cent passing 200-mesh as compared with less than 80 per cent passing 200-mesh for portland cement). Although some defects have been observed, the structure is still in service under unfavorable conditions.

Sand cements were used by the U. S. Reclamation Service for the construction of the Arrowrock, Elephant Butte, and Lahontan Dams, 1913-1916 (2). The active silica content of the sands which were interground with portland cement was small. Approximately equal parts of cement and sand were used. While some disintegration has occurred at the surface, recent investigations have shown the interior concrete of these structures to be sound.

In the Aberthaw tests on marine piles at the Charlestown Navy Yard, a German "iron" slag cement was used (4). It ranked well in the comparisons of resistance to sea-water exposure.

Bates, Phillips, and Wig included foreign puzzolans in a series of tests, reported in 1912, to determine the action of the salts in alkali water and sea water on cements (4). At the age of 3½ years, the specimens compared favorably with those for the various portland cements. In 1933, Bates stated that "the puzzolanic cements have the outstanding properties of low early strength, but very steady gain in strength, and an excellent resistance to aggressive waters after they have acquired a certain part of their ultimate strength." (11).

In 1924, Atwood and Johnson published a comprehensive analysis of the results obtained from the use of concrete in sea water (4). Based upon an extended search of the literature, they stated:

"1. Practically all skilled experimenters with hydraulic binding agents, for the last 100 years, have agreed that the primary cause for the disintegration of mortar and concrete in sulphate-carrying waters, such as sea water and many alkali waters, is the attack on the free lime in the mortar by the sulphates of the water.

"2. The majority of the authorities agree that this disintegration can be prevented by the addition to standard portland cement of a properly constituted siliceous material, which, by combination with the free lime released in the process of setting, will form a cementing material insoluble in sulphate-bearing water."

Hughes and Levens conducted a series of tests (1930) to determine the strength and shrinkage of mortars made with blends (not interground) of each of two portland cements with Fresno pumicite, ground blast-furnace slag, ground Haydite, and a volcanic ash (9). It was found in general that when an equal weight of puzzolan was substituted for portland cement (a) more water was required for equal consistency, (b) the compressive strength was lower at 7 days but higher at 90 days and thereafter, (c) greatest relative strengths were for puzzolans of greatest activity, (d) the shrinkage was greater for all blends except those containing ground Haydite, and (e) the quantitative effect was different for the two portland cements.

The Los Angeles County Flood Control District (1930) has conducted strength and other tests of concretes containing siliceous materials used as a replacement (not interground) for portland cement. It was found that the materials of greatest activity were tuff, pumicite, and diatomaceous earth. The most favorable results were obtained with about 20 per cent (by weight) of tuff or pumicite, and with a smaller percentage of diatomaceous earth. When replacement of cement was made by volume instead of by weight, the strength and watertightness of concretes containing puzzolan-portland cement were generally less than for portland cement alone.*

*Owing to differences in specific gravity, replacement of 20 per cent of the cement by an equal *volume* of pumicite or tuff amounts to the substitution of 12 pounds of puzzolan for 20 pounds of portland cement.

Blending part pumicite and part diatomaceous earth was superior in some respects to blending any single puzzolan.

For several years, preliminary tests bearing on the suitability and use of high-silica cements have been carried on in the Engineering Materials Laboratory of the University of California, these tests pointing toward a comprehensive investigation which would include the major significant variables. The results of these preliminary tests are given later herein.

INTERPRETATION OF PAST EXPERIENCE AND RESEARCH

It is extremely difficult to interpret the results of past experience and research in the use of blended cements, for several reasons. There is a great variety of siliceous materials, differing in chemical composition and (even for the same chemical composition) differing in physical structure; therefore statements regarding one type of silica may not apply to other types. Although the type of portland cement greatly influences the effectiveness of siliceous admixtures, but little information regarding the cement can be found in the literature bearing on the subject. The conditions of test vary widely in the different countries, and the field conditions of curing, climate, and exposure vary from place to place. But few tests have been made to determine properties other than the tensile or compressive strength and the resistance to exposure in sea water or alkali solutions. However, an attempt to summarize the present status of information is made in the following statements:

1. It is generally agreed that during the hydration of portland cement "free lime" is formed as calcium hydroxide, which is soluble. If exposed to the action of water, the lime in excess of that which becomes carbonated by exposure to air may leach out, increasing the porosity of the mass. If exposed to the action of sea or alkali water, the free lime is attacked to a greater or less extent depending upon the porosity of the concrete, and compounds of larger volume are formed which tend to disrupt the mass.

2. The majority of authorities agree that properly constituted siliceous material combines with the free lime released in the process of hydration, forming an insoluble cementing material.

3. That siliceous materials be effective in this respect (puzzolanic), they must be finely divided and intimately mixed with the portland cement, and must contain "active silica."

4. The activity of a siliceous material may in some cases be increased by calcination at even a relatively low temperature, probably by changing the material from a partly crystalline form to the more active amorphous state.

5. The optimum amount of puzzolan varies from 10 to 30 per cent by weight, depending chiefly upon the chemical composition of the cement.

6. Where the puzzolan is used as an *addition* to portland cement, for a given consistency the water requirement and the workability of mortar or concrete is increased. Where the puzzolan is used as a *replacement*, the water requirement may or may not be increased, depending upon (a) whether the replacement is by volume or by weight, and (b) whether the puzzolan is diatomaceous or solid-grained, less additional water being required by a solid-grained puzzolan. The increased fineness due to intergrinding tends to lessen the water requirement for concrete but to increase it for neat pastes. Accompanying the greater workability of some puzzolan-portland cements is a tendency for the concrete to stick to the working tools and the forms.

7. The strength of concretes containing puzzolans as a replacement for portland cement is generally at the early ages less than, and at the later ages is approximately equal to, the strength of straight portland-cement concretes.

8. Comparative tests for durability as evidenced by resistance to sea-water or alkali-water exposure indicate the greater durability of puzzolan-portland cements as compared with corresponding straight portland cements.

9. Volume-change tests indicate that when the curing period is short, a puzzolan-portland cement exhibits greater shrinkage upon drying than a normal portland, but that the amount of this shrinkage may be less than that for a low-lime portland cement. It appears that solid-grained puzzolans exhibit less shrinkage than do diatomaceous puzzolans. For puzzolan-portland cements, conditions favorable to low volume change are (a) calcination of the puzzolanic material, (b) high lime content of the portland-cement clinker, and (c) long-continued moist curing or high-temperature moist curing.

10. The rate and total amount of heat of hardening of puzzolan-portland cements is less than that of normal portland cements. The reaction between the silica of the puzzolan and the lime of the portland cement contributes only a small amount to the heat of hydration.

11. Permeability tests indicate that where equal weights are used, puzzolan-portland cements exhibit a greater degree of watertightness in mortar and concrete than do portland cements.

12. The cost of manufacturing a puzzolan-portland cement in large quantity may be less than for a portland cement, provided that the puzzolan is available at low cost.

13. Under the condition of steam curing, puzzolan-portland cements exhibit relatively high early strength and low volume changes, in comparison with portland cements moist cured at normal or mass-concrete curing temperatures.

14. Under conditions such as exist in mass concrete, the indications are that at the later ages the strength for puzzolan-portland cements may be as great as for normal portland cements, the volume changes may be as small, and the resistance to the action of weather is satisfactory.

15. Puzzolan-portland cements are consistently sound.

16. The unit weight of concrete containing puzzolan-portland cements is slightly (generally less than 1 per cent) less than that of concrete containing the same proportion of portland cement.

17. Puzzolan-portland cements exhibit somewhat lower elastic moduli in mortar and concrete than do portland cements.

18. It is indicated that grinding to high fineness is more economical for puzzolan-portland cements than for portland cements, especially if (a) a high-lime portland cement is used in the blend, and (b) the blended cement is compared with a low-lime portland cement of comparable characteristics as regards strength, heat of hydration, etc.

RESEARCH PROGRAM AT THE UNIVERSITY OF CALIFORNIA

During the last three years, the following investigations on high-silica cements have been in progress in the Engineering Materials Laboratory of the University of California.

1. Low-lime portland cements.—Studies of cements of high dicalcium-silicate content, included in the cement investigations for Boulder Dam. Determination of possibilities of unblended portland cements with regard to heat of hydration, strength, volume changes, resistance to weathering action (herein termed "durability"), and resistance to action of sodium sulphate solutions, in which last regard the use of low-lime cements appears to be especially favorable. Description of testing procedure and principal test results published (12, 13).

2. Pumicite-portland cements—Effect of adding Fresno pumicite to high-lime, normal-lime, and low-lime portland cements upon volume changes, compressive strength, and heat generation of mortars or neat-cement pastes. Variables included (a) calcination temperature of pumicite, (b) chemical composition of cement, (c) curing conditions, (d) method of blending (adding vs. intergrinding), and (e) proportion of pumicite to cement. Description of testing procedure and major test results published (14).

3. Monterey shale-portland cements. -Effect of intergrinding a calcined Monterey (diatomaceous) shale in various percentages with a normal-lime and a low-lime portland cement, with the addition of lime or lime hydrate, upon heat of hydration, strength, durability, and volume changes. Included extremely low-heat cements for mass-concrete construction.

4. Activity of various puzzolans.—Determination of ability to combine with lime of uncalcined and calcined pumicite, Monterey shale, diatomaceous earth, clay,

tuff, granite, quartzite, and of calcined oil-impregnated diatomaceous earth. Tests for compressive strength of silica-lime-sand mortars.

5. 1934 Investigation.—Comprehensive tests of cements blended by intergrinding are now in progress, employing 60 combinations of siliceous materials and portland cements. Details of the program are given later herein.

DISCUSSION OF TEST RESULTS

While it is not possible in this paper to present all of the results of tests on high-silica cements made at the University of California, the more significant results are here either presented or discussed, in order of the various investigations as previously listed.

Pumicite-portland cements.—A series of tests was made on 25 portland and blended pumicite-portland cements (14). High-lime, normal-lime and low-lime portland-cement clinkers were employed, all of which were low in alumina content. The pumicite and cement were blended both by simple addition and by intergrinding (replacement by weight).

The pumicite is a volcanic ash from a natural deposit near Fresno, California. It occurs finely divided, with 96 per cent passing the 325-mesh sieve. Its principal constituents are 75 per cent silica, 15 per cent alumina, and 1 per cent iron oxide. It was used both uncalcined and calcined.

In Table 1 are given selected values illustrating the relation between heat generation and mortar compressive strength for plain and blended cements. Comparing the normal-lime portland cement and the same cement blended with 20 per cent of pumicite, it is seen that both the heat of hydration and the compressive strength are lower for the blended cement, but that the percentage of difference in heat is greater than that in strength. While the strength of the 30 per cent blend with normal-lime clinker is reasonably high (4710 p.s.i. at 1 year), it is apparent that under the conditions of these tests 30 per cent of pumicite provides more than enough active silica to combine with the lime liberated by the cement.

TABLE 1—RELATION BETWEEN HEAT GENERATION AND MORTAR COMPRESSIVE STRENGTH OF PUMICITE-PORTLAND CEMENTS

Clinker	Percentage of Pumicite[1]	Heat of Hydration, Cal. per Gram		Compressive Strength of Mortar, p.s.i.[2]			
		7 Da.	28 Da.	7 Da.	28 Da.	6 Mo.	1 Yr.
Normal-lime (55% C_3S)	0	84	94	3860	5350	5430	5260
	20	68	77	2370	4790	4950	5270
	30	60	69	2350	3970	3870	4710
Low-lime (33% C_3S)	0	67	79	2250	4270	5140	5060
High-lime (70% C_3S), low-alumina	0	81	93	3670	4510	5040	5280
	30	62	74	2100	3970	4360	——

[1]Replacement of cement by weight; pumicite calcined; cements blended by intergrinding.
[2]Strength specimens cured in sealed cans 2 days at 70° F., 26 days at 100° F.; cans removed at 28 days; thereafter specimens stored under water at 70° F. (*C* curing).

Comparing cements of nearly equal 28-day heat generation, which is a rational basis for mass-concrete work, it is seen that the blend of 20 per cent of pumicite with the normal-lime clinker produces less heat than the straight low-lime cement, and equal or higher mortar strength at all ages up to 1 year. Further, the blends of 30 per cent of pumicite with the high-lime and the normal-lime clinkers produce less

TABLE 2—MORTAR COMPRESSIVE STRENGTH OF PORTLAND AND PUMICITE-PORTLAND CEMENTS UNDER CONTINUOUS MOIST-CURING CONDITIONS

Clinker	Percentage of Pumicite[1]	Water-Cement Ratio, by wt.	Compressive Strength of Mortar, p.s.i.[2]					
			3 Da.	7 Da.	28 Da.	6 Mo.	1 Yr.	21 Mo.
High-lime (70% C_3S)	0	0.57	2290	3340	5530	6830	7070	—
	30	0.59	1550	2440	3540	5690	7010	—
Normal-lime (55% C_3S)	0	0.53	1730	2150	3870	5080	5310	4990
	20	0.57	970	1520	2450	4360	5030	5430

[1]Replacement of cement by weight; pumicite not interground.
[2]Curing under moist conditions at 70° F.

heat than the straight low-lime cement, with reasonably high strengths in comparison.

Studies have been made of the effect of variations in curing conditions upon the puzzolanic action of pumicite blended with high-lime, normal-lime, and low-lime portland cements (14). Under continuous moisture-curing conditions (either continuously at 70° F. or at 100° F. up to the age of 28 days and at 70° F. thereafter), the mortar compressive strength for the pumicite-portland cements is at the age of 1 year approximately equal to that for the corresponding straight portland cements. However, under the condition of storage of the specimens (2 by 4-in. cylinders) in dry air after 28 days, the strength for the blended cements never approaches that for the straight cements.

In Table 2 are given values of mortar compressive strength for two portland cements, both plain and blended with pumicite. It is seen that under the moist curing conditions of the tests, the strengths at later ages are approximately equal, even though additional mixing water was used with the blended cement.

Volume-change data for a number of portland and pumicite-portland cements are presented in Table 3. The measurements were made on 1½ by 1½ by 12-in. mortar bars, therefore both the actual values and the differences between cements are much greater than for corresponding concretes. While the space available does not permit a detailed discussion of these results, it is significant that when straight and blended cements of approximately equal heat generation and strength are compared, the volume changes are also approximately equal.

Tests were made to determine the effect of steam curing upon the compressive strength and volume changes of concretes containing a high-lime portland cement and the same cement blended (not interground) with 20 per cent of pumicite (replacement of cement by weight). The cement-aggregate ratio was 1:5 by weight, using 0 to ⅜-in. gravel. After 16 hours of moist curing at 70° F. the specimens were cured (in the molds) for 8 hours in steam under pressure of 100 p.s.i. at 336° F.; they were then slowly cooled to normal temperature and were stored in air of 50 per cent relative humidity at 70° F. up to the age of 28 days. During the period from 28 to 35 days the specimens were stored under water at 70° F., then were returned to dry storage. The pumicite-portland cement exhibited higher strength than did the portland cement up to the age of 28 days, even though it required 18 per cent more mixing water for equal consistency. However, under the particular condition of steam curing, the strength for both cements was at the age of 28 days considerably lower than that for the straight portland cement cured continuously moist at 70° F. Under the condition of steam curing the pumicite-portland cement and the portland cement

TABLE 3—VOLUME CHANGES OF MORTARS CONTAINING PORTLAND CEMENTS AND PUMICITE-PORTLAND CEMENTS

Clinker	Percentage of Pumicite[1]	Water-Cement Ratio, by Wt.	Curing Conditions[2]			Change in Length, Millionths Per Unit				
			Group	2 to 28 Days		Expansion During 28-Day Curing[3]		Contraction from Length at 28 Days, Storage in Air of 50% Rel. Hum. at 70°F.		
				Moisture	Temp., °F.	7 da.	28 da.	35da.	6 mo.	1 yr.
Normal-lime (55% C_3S)	0	0.55	A	Sealed	100	47	83	184	661	—
	0		E	Fog	70	33[a]	65	418	947	1001
	20	0.55	A	Sealed	100	51	106	254	831	847
	20		E	Fog	70	26	50	481	914	940
	30	0.57	A	Sealed	100	77	130	99	844	887
	30		E	Fog	70	18	50	467	975	1003
	40	0.56	A	Sealed	100	52	129	22	740	—
	40		E	Fog	70	—	60	432	969	1007
Low-lime (33% C_3S)	0	0.57	A	Sealed	100	59	102	311	776	790
	0		E	Fog	70	45	75	587	1113	1127
	30	0.57	A	Sealed	100	85	147	49	941	1019
	30		E	Fog	70	68	97	593	1187	1215
High-lime (70% C_3S)	0	0.54	A	Sealed	100	39	54	276	664	684
	0		E	Fog	70	32	38	328	875	901
	30	0.56	A	Sealed	100	48	102	135	775	815
	30		E	Fog	70	24	31	356	869	917

[1]Replacement of portland cement by weight; pumicite interground with cement.
[2]Specimens of groups A and E cured in molds in fog room 2 days; from 2 to 28 days as indicated in table; after 28 days stored in air of 50 per cent relative humidity at 70°F.
[3]Initial reading taken at 2 days.
[a]At 11 days.

NOTE 1—Specimens are 1½ by 1½ by 12-in. mortar bars, cement-aggregate ratio 1:3 ¼ by weight, using O-No. 4 sand.
NOTE 2—Pumicite calcined to incipient fusion at 2200°F.

exhibited almost equal volume changes, and these volume changes were considerably less than those for the straight portland cement cured continuously moist at 70° F.

Following are the principal findings of the preliminary tests on pumicite-portland cements:

1. The heat of hydration of a pumicite-portland cement is less than for the corresponding straight portland cement, but the percentage of reduction in heat is less than the percentage of pumicite.

2. The compressive strength of mortar at early ages is shown to be lower for all pumicite-portland cements than for corresponding straight portland cements, but with the proper proportion of pumicite and continued moist curing, the strengths are substantially equal at later ages. From 20 to 30 per cent (replacement by weight) of pumicite may be blended with a normal-lime cement without reducing the mortar strength at the age of 1 year.

3. A long period of moist curing is necessary to develop the potential strength of a pumicite-portland cement.

4. High-pressure steam curing of the type herein considered results in higher strength of concrete for pumicite-portland cement than for corresponding straight portland cement.

5. Comparative tests of *concretes* show higher relative strengths for the blended cements than the tests on *mortars*.

6. The elastic modulus is consistently lower for the pumicite-portland cements, but the difference diminishes at the later ages as the strengths approach equality.

7. When mortar is cured for a preliminary period of 28 days at 100° F., the subsequent contraction upon drying is shown to be lower at the early ages and higher at the later ages for pumicite-portland cement than for straight portland cement.

8. When mortar is cured for a preliminary period of 28 days at 70° F., the subsequent contraction upon drying is shown to be at all ages almost equal for the pumicite-portland cement and for the straight portland cement, except when the percentage of pumicite is considerably greater than the optimum for the particular clinker used.

9. For mortars cured under mass-concrete conditions for 28 days and stored under water thereafter, the expansions exhibited by pumicite-portland and straight portland cements are approximately equal.

10. High-pressure steam curing reduces greatly the subsequent volume changes of concretes for pumicite-portland and straight portland cements, the volume changes being approximately equal for the two types of cements.

11. Pumicite-portland cements and low-lime straight portland cements of equal fineness exhibit at all ages approximately equal heats of hydration, strengths, and volume changes although the actual silica content of the pumicite-portland cements is 50 to 75 per cent greater than that of the low-lime straight portland cements.

Monterey Shale-portland Cements.—A series of tests was made on 8 portland cements and blended cements made by intergrinding normal-lime and low-lime portland cements with Monterey shale, with lime added either in the form of calcium oxide or calcium hydrate. The Monterey shale was of diatomaceous origin, occurring near Santa Cruz, California, and consisting chiefly of about 82 per cent silica and 10 per cent alumina.

TABLE 4—HEAT OF HYDRATION AND MORTAR COMPRESSIVE STRENGTH—MONTEREY SHALE-PORTLAND CEMENTS

Clinker	Percentage of Shale[a]	Per Cent Pass 200-Mesh	Water-Cement Ratio, by Wt.	Heat of Hydration (Mass Curing) cal./gram		Compressive Strength of Mortar, p.s.i.							
						Mass-Concrete Curing				70° F. Curing			
				28 Da.	3 Mo.	7 Da.	28 Da.	3 Mo.[b]	1 Yr.[b]	7 Da.	28 Da.	3 Mo.	1 Yr.
Normal-lime (commercial)	30	97.9	0.61	90	92	4090	4100	4430	4320	2870	4190	4880	5060
	30	93.0	0.65	83	84	3040	3550	3680	3550	1950	3500	4640	5060
Low-lime, lowalumina	0	95.6	0.58	67	76	750	2230	3410	——	590	1560	3580	5500
	50	97.7	0.65	44	56	1040	2430	2660	3070	650	1850	3150	4120
Avg. of 25 commercial portland cements	0	87.9	0.58	95	99	3050	3990	4180	4760	2320	3570	4500	5160

[a]Replacement of cement by weight; blending by intergrinding; lime (5 per cent of total cement) added in the form of calcium oxide.
[b]Curing at 70° F. after 28 days.

In Table 4 are given values of heat of hydration and mortar compressive strength for four straight and blended cements. The average results for 25 commercial portland cements are also given, for comparison. The water requirement of the shale-portland cements is greater than that of the portland cements, although the blended cements are considerably finer and ordinarily finer cements require less water.

The question has often arisen as to the minimum practicable heat of hydration consistent with a reasonably high strength, obtainable with cements for mass-

concrete construction. In this connection, the values in Table 4 for the low-lime portland cement blended with 50 per cent of shale are of interest. It is seen that at the age of 3 months, the heat of hydration for the blended cement is 57 per cent of that for the average of 25 commercial portlands, but that the strength under mass-curing conditions is 64 per cent and under 70° F.-curing conditions is 70 per cent of that of the portland cements. This heat of hydration (56 cal. per g. at 3 mo.) is lower than was exhibited by any portland cement included in the Boulder Dam cement investigations which covered a wide range of chemical compositions. Furthermore the early strength was several times greater than that for the low-lime straight portland cement, of the Boulder Dam cement investigations, which gave the lowest heat of hydration; at the age of one year the strengths were substantially equal. A cement having an extremely low heat of hydration such as that exhibited by this shale-portland cement would be necessary to prevent the cracking, due to thermal change, of a large mass of concrete constructed without contraction joints.

The expansion of mortars containing this low-heat blended cement was, under the condition of continuous moist curing, materially less than for an average portland cement. However, the contraction upon drying was considerably greater than for an average portland cement.

In Table 5 are given values of the flexural and compressive strength of concretes containing several standard portland, low-heat portland, and shale-portland cements. The cores tested were cut from outdoor slabs exposed to normal weather conditions; the cylinders and beams were cured continuously under moist conditions at 70° F. The strengths for the shale-portland cement compare favorably with the average for the low-heat and even the standard portland cements. Further, for the shale-portland cement, the compressive strength of the cores is materially higher than that of the cylinders.

TABLE 5—FLEXURAL AND COMPRESSIVE STRENGTH OF CONCRETE—STANDARD PORTLAND, LOW-HEAT PORTLAND, AND SHALE-PORTLAND CEMENTS

Cements		Specific Surface, Sq. Cm. per Gram	Modulus of Rupture at 28 Days, p.s.i.	Compressive Strength of Concrete,[1] p.s.i.			
				28 Days		1 Year	
				Cylinders	Cores	Cylinders	Cores
Standard portlands	Max.	1570	1053	6170	6310	7310	7430
	Min.	1230	807	3760	4380	5760	5760
	Avg.	1360	945	4870	5390	6370	6500
Low-heat portlands	Max.	1740	1006	4920	5570	6640	6930
	Min.	1420	872	3940	4870	5140	5650
	Avg.	1570	938	4480	5210	5850	6120
Shale-portland		1990	926	4430	5060	5300	6050

[1]Cement-aggregate ratio approximately 1:6 by weight, using 0 to 1½-in. sand-gravel aggregate; water-cement ratio approximately 0.47 by weight.

Other tests indicate that shale-portland cements are more resistant to solutions of sodium sulphate than are normal portland cements.

The results of the preliminary tests on Monterey-shale portland cements may be summarized as follows:

1. The water requirement for a given consistency of concrete is approximately 6 per cent greater for the shale-portland cements.

2. The heat of hydration of all the shale-portland cements tested is less than that of the corresponding straight portland cements, but the percentage of difference in heat of hydration is not as great as the percentage of cement replacement.

3. For a blended cement containing 30 per cent of Monterey shale and 70 per cent of normal-lime portland cement, the compressive strength is higher at early ages than that for the average of 25 commercial portland cements. This high early strength is probably due to the high fineness (large surface area) of the blended cement. At later ages the strength is approximately equal to that of the average of 25 portland cements, except that when the curing condition was that of mass concrete for 28 days followed by storage at 70° F. there is little gain in strength for the shale-portland cement between the ages of 28 days and 6 months.

4. The expansion of mortars cured continuously under water at 70° F. following a period of curing under mass-concrete conditions for 28 days is less for shale-portland cements than for straight portland cements.

5. The contraction upon drying, following a period of curing either under mass-concrete conditions or at 70° F. for 28 days, is greater for shale-portland cements than for corresponding straight portland cements. The difference in contraction is far less for concretes than for mortars (10 per cent *vs.* 90 per cent).

6. Results of durability tests (alternate freezing and thawing and alternate wetting and drying, followed by a test for compressive strength) on blended cements containing 30 per cent of shale compare favorably with the average results of similar tests made on commercial portland cements.

Activity of Various Puzzolans.—In Table 6 are presented the results of tests to determine the activity of nine siliceous materials, as indicated by the compressive strength of silica-lime-sand mortar at the ages of 7 and 28 days. It is seen that there is a wide range in activity as shown by these tests, the strengths at the age of 28 days varying from 120 to 1700 p.s.i. The strengths to be expected from these materials when blended with portland cements may not be in the same order as indicated by these tests, although the materials of low activity (as granite, quartzite, and Ottawa sand) will undoubtedly contribute less to strength than the materials of high activity (as Monterey shale and diatomaceous earth). The effect of calcination upon the activity of the various materials may be seen by direct comparison in Table 6.

Optimum Proportions of Puzzolan and Lime.—In a series of compression tests on puzzolan-lime-sand mortars (puzzolan and lime interground), it was found that the highest strength is obtained when approximately the following proportions are used:

Puzzolan	Per Cent, by Weight	
	Puzzolan	Lime
Calcined pumicite	80	20
Calcined Monterey shale	70	30
Calcined diatomaceous earth	60	40

From these tests, it appears that there is a relation between either the density or the surface area of the puzzolan and the amount which should be used.

1934 INVESTIGATION

Tests of cements blended by intergrinding are now in progress, employing 60 combinations of siliceous materials and portland cements, to determine the effect of the following factors:

a. Chemical composition of siliceous material,
b. Physical character of siliceous material (solid-grained *vs.* diatomaceous, etc.),
c. Percentage of siliceous material used (for each type of portland cement),

TABLE 6—ACTIVITY OF VARIOUS SILICEOUS MATERIALS AS INDICATED BY COMPRESSIVE STRENGTH OF SILICA-LIME-SAND MORTARS

Siliceous Material	Calcined (C) or Uncalcined (U)	Specific Surface of Siliceous Material, Sq. Cm. per Gram	Ratio of Water to Cementing Material, by Weight[a]	Compressive Strength, p.s.i.[b]	
				7 Da.	28 Da.
Monterey shale	U	2260	1.00	1540	1700
	C	2330	0.95	1400	1520
Casmalia earth[c]	C	2300	0.90	1640	1530
Diatomaceous earth	U	3140	1.38	920	1010
	C	2830	1.10	1340	1420
Clay	U	1700	0.90	480	740
	C	1520	0.65	890	1120
Pumicite	U[d]	2150	0.66	640	780
	C[d]	1750	0.69	490	600
Tuff No. 1	U	2110	0.65	Note[e]	80
	C	2120	0.73	510	600
Tuff No. 2	U	2080	0.93	550	570
	C	2300	0.83	570	710
Granite	U	2060	0.58	50	200
	C	1990	0.58	50	240
Quartzite	U	1860	0.62	Note[e]	170
	C	1610	0.56	Note[e]	190
Ottawa Sand	U	1890	0.59	Note[e]	120

[a]Required to give a mortar flow of 150 per cent.
[b]Mortar mix 1 part, by weight, of cementing material to 3 parts of 0 to No. 4 sand; cementing material consists of 1 part, by weight, of hydrated lime to 2 parts of oven-dry siliceous material.
[c]Diatomaceous earth naturally impregnated with oil; calcined to remove oil and carbon.
[d]Pumicite in natural state, not ground.
[e]Insufficient strength for test.
Notes—All samples except pumicite ground in ball mill for equal periods.
Specimens are 2 by 4-in. cylinders, molded in metal containers which are sealed as soon as filled.
Specimens are cured for 12 hr. at 70° F., then for 12 hr. at 100° F., then at 130° F. until 12 hr. before testing, when the temperature is reduced to 70° F.

d. Fineness of siliceous material before intergrinding,
e. Calcination of siliceous material,
f. Final fineness of blended cement, and
g. Chemical composition of portland-cement clinker (including effect of adding lime separately to the blend)

upon the following properties:

A. Compressive strength under various curing conditions,
B. Resistance to the action of weather and of sulphate waters,
C. Volume changes under various curing conditions,
D. Heat of hydration,
E. Porosity or absorption,
F. Water requirement for given consistency.
G. Tensile strength,
H. Soundness,
I. Setting time,
J. Unit weight,
K. Thermal expansion,
L. Modulus of elasticity, and
M. Ease of grinding.

In planning the 1934 investigation, the information from the earlier tests and from the tests of other investigators has been considered, particularly with regard to establishing the probable range of favorable

calcination temperatures, percentages of the various siliceous materials, grinding conditions, and curing temperatures. It has been considered that blended puzzolan-portland cements may be adapted to a large variety of construction including mass concrete, sea-water construction, works exposed to alkali waters or soils, highways, and precast concrete products; and the information sought is such as will aid in the preparation of specifications for blended cements. Since puzzolanic materials are obtainable from a variety of sources, it will be necessary to establish some method of determining the suitability of various deposits. Further, it has been considered that materials not now considered as puzzolanic may be rendered so by proper calcination.

Nine of the siliceous materials are listed in Table 6; other siliceous materials and limestone are also under consideration. The silica content ranges from 62.0 to 99.9 per cent, and the alumina content ranges from 0 to 21 per cent. High-lime, normal-lime, and low-lime portland cement clinkers are employed. The percentage of cement replacement ranges from 10 to 45. In general, each puzzolan is employed both in the natural state (crushed if necessary) and after calcination. Sufficient calcined gypsum is added to make the SO_3 content of the plain cement equal to 1.5 per cent. In some cases, freshly calcined high-calcium lime is added in the proportion of 1 part (by weight) of lime to 4 parts of puzzolan. During the grinding, sufficient water is added to insure the hydration of the lime without excess water. For use in test specimens, all cements are ground in a batch mill for equal periods.

The tests completed to date include only the chemical analysis and determinations of (1) relative solubility in hydrochloric acid, (2) relative activity as determined by compression tests of silica-lime-sand mortars (see Table 6), and (3) relative grindability of certain cements and all siliceous materials (for fineness after grinding for equal periods see Table 6). These data are to be considered in connection with the results of tests for strength, volume changes, durability, heat of hydration, etc.

For such discussion of this paper as may develop, readers are referred to the JOURNAL *for October, 1934 (Proceedings, Vol. 31). Discussions should be available to the Secretary by August 1, 1934.*

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Observations on European Practice in Concrete Design and Construction*

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In a recent report to the Society for the Promotion of Engineering Education, Prof. W. E. Wickenden, President of the Case School of Applied Science, stated—

> The debt of engineering education in America to Europe for early models is well set forth in Dr. Mann's report. Later evolution has been along lines which clearly reflect the characteristic contrasts in the economic and educational life of the two continents. . . . In Europe men have been relatively many and raw materials relatively scarce. . . . Engineering in Europe has felt a natural urge toward refinement of design, approaching close to theoretical limits, in its concern for economy of materials, but there has been little urge toward economy of labor. America has always been and still is a land of abundant space, cheap and widely distributed raw materials, and relatively scant but highly mobile population. Cheap materials and costly labor have been dominant influences in our industrial life. The life of the frontier bred up a race of self-reliant men, schooled in the art of ingenious contrivance rather than scientific refinements. The work of these natural innovators displayed a certain boldness of execution and of clever dexterity in replacing human labor rather than theoretical refinement or economy of design.
>
> It may be conceded that formal attainments, especially in mathematical theory and refined design have been superior in the better schools of Europe. . . . Granting all that has been claimed for the formal superiority of European education, although it has probably been overstated, it may still be true that American policies and standards have been vastly more advantageous to the economic development of the nation than a close adherence to European models might have been. At least, we have no grounds for undertaking a comparison of our experience with that of Europe under the spell of an inferiority complex. An attitude of superiority would be equally amiss.

I quote these statements for two reasons; first, because the reasons for the principal differences in concrete design and construction practices of Europe and of the United States can be better understood in the light of the conditions so well described by Professor Wickenden; and second, because Professor Wickenden's last statement, relative to the comparative merits of engineering education, is, in my opinion,

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equally applicable to engineering practice. The statements contained in this paper are, therefore, not to be taken as implying invidious comparisons between American and European practices, to the detriment of the one or of the other.

The title of this paper is somewhat misleading. My studies abroad were confined almost entirely to French engineering. This paper is, therefore, devoted largely to a resumé of French practice in concrete design and construction. While I visited other countries in Europe, I did not have an opportunity to become familiar with their practices.

No one can justly deny our debt to the French for the invention and early development of reinforced concrete. The work of Vicat and Le Chatellier on cements, and that of Monnier, Hennebique, Considère, Levy, Mesnager, Feret, Freyssinet, Caquot, and others on design and construction, merits our grateful appreciation.

The first reinforced concrete structure was a small boat exhibited at the Paris Exhibition of 1855 by Lambot. The use of reinforced concrete developed rapidly in France, and in 1900 the Minister of Public Works appointed the first "Commission on Reinforced Concrete," to establish regulations for design and construction. This Commission submitted its report in January, 1906 and it was approved by the Minister of Public Works in October, 1906. As far as I know, the work of this Commission comprises the first extensive series of tests of concrete and reinforced concrete and is the first organized effort to regulate design and construction procedures. The report is very complete, being contained in a volume of 481 pages. It antedates the first final report of our "Joint Committee on Standard Specifications for Concrete and Reinforced Concrete," by ten years, although progress reports of that Committee appeared in 1909 and 1912.

The regulations of 1906 are still in effect in France. While mandatory only for public works, they are actually followed by practically all engineers and are accepted as a basis for expert testimony by the courts.

In general, French design practice tends toward greater refinement in theoretical analysis and in attention to minor details than ours. It appears, also, that the French have great confidence in the applicability of refined theory to actual structures. This results in the use of light sections involving considerable form work and a minimum of those stiffening members, which, theoretically, perform no useful work. As a result, they achieve economy in materials, usually with an increase in labor. This practice appears to be justified by economic

conditions. To illustrate—on a large reinforced concrete project being constructed in 1932 just outside of Paris the costs were as follows:

Cement	$ 1.37 per bbl., net
Gravel	1.23 per cu. yd.
Sand	1.08 per cu. yd.
Steel	45.00 per ton
Common labor	16¢ per hour
Carpenters on form-work	24–30¢ per hour

When I inspected this work, I had just come from a naval station on our west coast where material costs were approximately the same as those listed above, but the labor costs were 56¢ per hour for common labor and 99¢ per hour for carpenters on form work. In normal times, there is approximately the same relationship between the pay of technical designers here and there. It is natural, therefore, that they should expend more time and effort on the design of a project and that the design should include details involving more meticulous work in the field in connection with building the forms, fabricating and erecting the reinforcement, and placing the concrete.

Two outstanding characteristics of French designers are individu--lity and ingenuity. Economic conditions have prevented extensive experimentation and testing comparable to what has been done in this country. As a result, the engineer relys more upon theoretical considerations than upon test results. This has lessened the tendency to standardize procedures and has encouraged individual initiative and ingenuity. The works of French engineers show many evidences of these influences.

On construction work, accurate proportioning, grading of aggregates, control of water, careful mixing, and adequate curing, are not given the attention which we attempt on our more important works.

In 1928, the "Syndicate of Reinforced Concrete Constructors" appointed a commission of eminent French engineers to prepare new regulations for concrete design and construction. The report of this commission was published in January, 1930, and the recommended regulations have been adopted by the Ministry of Air. The report is now under consideration by the Ministry of Public Works and it will probably be adopted by that Department to supersede the regulations of 1906. I will quote frequently from this report as illustrative of the best French practice.

It should be noted that in France most of the contracts for large works are based on "bidders design." This practice explains some of the procedures which are recommended by the commission.

MATERIALS

Reinforcing Steel

Mild steel, corresponding approximately to our structural grade, is usually preferred, but steels of higher yield point are coming into favor, especially for compression members. Freyssinet predicts the use of manganese-silicon steel with an elastic limit of 250,000 p.s.i. for lateral reinforcement of compression members. The steel is tested for fragility, yield point, ultimate strength, and elongation, and in cold bending. The yield point must not exceed 85 per cent of the ultimate strength. Only cold bending is permitted, except under very special conditions. When reinforcement of variable section is used (for example, in a built-up section the members of which are pierced by holes) the average fibre stress on the net section is arbitrarily increased by an amount depending upon the yield point of the steel. This increase varies from 10 per cent for mild steel to 20 per cent for hard steel. The working stress is a fixed percentage of the yield point stress. However, for steels having a yield point higher than 39,000 p.s.i., the standard percentage is not permitted unless special consideration is given to the effect of high stresses on the cracking of the concrete and to bond stress.

Only plain reinforcing bars are used in France. Débés states that if deformed bars are placed too close to the surface of the concrete, the deformations may cause spalling. Another prominent engineer stated that deforming the bars is believed to result in planes of weakness in the steel. Mesnager stated that he would use deformed bars if the French mills would make them.

Flat steel, 1.5 to 3 mm thick, is often used to tie compression steel and for shear reinforcement. This has the advantage of providing greater cover over the ties.

Cement

Portland cement is the standard. Special cements are permitted only for special uses and after "complete justification." There are several grades of portland cement, including high early strength portland, known as "Superciment."

"Ciment de laitier," i. e. slag cement, is widely used for parts of structures below ground water level. Hydraulic limes and natural cements are used, principally for mass work. Alumina cement is favored for exposure to sulphate waters, but its high cost (about 2½ times the cost of portland) has greatly restricted its use. A review of proposed French specifications for hydraulic cements is given in

the November-December 1933 issue of the Journal of the Institute.

Aggregate

This designation covers all of the material in the concrete mixture except the cement and water. No distinction is made between fine and coarse aggregate. The report of the commission states:

> The aggregate shall consist of grains graduated from 0.2 mm. to the maximum size and of a material which is resistant to climatological influence. The resistance of the aggregate must be superior to the resistance required of the concrete.

The maximum size of aggregate is ½ of the clear distance between bars for angular aggregate and ¾ of the distance for rounded particles.

> The granulometric composition shall be regular and constant and such as will result in a concrete of designed strength and with the desired consistency.

It is interesting to note the absence of definite rules for gradation; it is stated that the gradation should be determined experimentally, in the field, to provide the required strength with the specified cement factor. The commentary accompanying the commission's report states that gradation rules will probably be available in the near future.

Bolomey (a prominent Swiss Engineer) proposes the following formula for the grading curve, with the cement included,

$$P = A + (100 - A)\sqrt{\frac{d}{D}}$$

where, D is the maximum size of aggregate, *m.m.*
d is the diameter of a particular particle, *m.m.*
P is the percentage by weight of sizes smaller than d
A is a factor depending upon the character of the aggregate and the degree of plasticity of the concrete.

For gravel, A varies from 10 for dry concrete to 12 for very plastic mixtures; for crushed rock, A varies from 12 to 14.

Concrete

> The concrete shall be made of a homogeneous mixture of cement, water, and aggregate. . . . The concrete shall have a plasticity such that it will thoroughly envelop the reinforcement and will be easily molded into the forms. More plastic mixtures may be used provided allowance is made for the corresponding diminution in density and strength. Stiffer mixtures may be used only if specially powerful means of placing are applied.

It should be noted that there is no specific requirement for maximum water content. The commentary states that extra water may be used as a "vehicule" provided it is subsequently removed by compacting the mass. Reliance is placed on the job strength tests to determine the quality of the concrete and the regulations emphasize the importance of job tests under field conditions.

While the French recognize the relationship between the water-cement ratio and strength, they believe that the difficulty of accurate determination of the water content of the concrete *in its final state* is such as to vitiate the value of this ratio as a means of predetermining strength.

Freyssinet states that no relationship necessarily exists between the quantity of *mixing water* and the strength, and that such a relationship can only exist under very rigidly controlled conditions, such as would not exist on ordinary work. He states that the strength of the concrete is dependent upon the quantity of water *finally* retained in the mixture, after all air voids have been eliminated. In most mixtures, with energetic compacting, excess water is brought to the surface and the strength of the concrete is greater than would be indicated by the initial water ratio. The possibilities of absorption of water by the forms and loss of water by leakage are mentioned.

The regulations state:

To obtain uniformity in the water content, tests to determine the plasticity by lowering of a definite geometric volume can be advantageously used.

This refers to our standard slump test. They state that:

the amount of mixing water cannot be accurately determined on the job by direct measurement because of the variation of the water content in the aggregate. The test of plasticity is preferable, provided it is made with concrete representative of the average of the mixture.

Débés has proposed the following modified version of the water cement ratio-strength law:

The strength of a workable concrete at a given age and made with a given cement varies inversely with the ratio between the *quantity of water which remains incorporated in the concrete after it has been compacted*, and the quantity of cement.

Bolomey proposed the following formula:

$$Y_2 = K\left[\left(\frac{\delta}{2.35}\right)^2 + \frac{C}{E}\right]^{2/3}$$

where, Y_2 = unit strength of concrete in Kg/cm^2
C = weight of cement (Kg.)
E = weight of water (Kg.)
δ = density of aggregate in loose condition
K = factor dependent upon the character of the cement and the age of the concrete.

This formula indicates the advantage of using large aggregate with greater density, when conditions permit. It takes into account, also, the characteristics of the cement, including the age-strength relationship.

French specifications usually require a definite weight of cement per cubic meter of concrete in place; this is called the "dosage," that is,

the cement factor. The regulations require that the dosage shall be determined by job test, consideration being given to required strength, character of the aggregate, period of curing, quality of the cement, and desired consistency. No definite dosages are specified, but for reinforced concrete a minimum dosage is fixed. This minimum is given by the following formula:

$$C = \frac{550}{\sqrt[4]{g}}$$

where g is the maximum size of the aggregate in millimeters, and C is the kilograms of cement per cubic meter of concrete. For 1 inch aggregate this would be equivalent to 4.4 sacks of cement per cubic yard of concrete.

The regulations prescribe that the strength of the concrete shall be determined by job tests sufficient in number to give the desired accuracy. The commentary states:

The experience of all good builders has shown the necessity of these tests, which, in the last analysis, alone permit the determination of the quantities of aggregate and cement and the consistency to be used.

The test procedure requires that the tensile strength of the concrete be determined by a flexural test on a square prism with lateral dimension approximately three times the maximum size of the coarse aggregate, and length four times the lateral dimension. The tensile strength is taken to be

$$f_t = \frac{3.6M}{b^3}$$

where M is the bending moment, which is uniform throughout the length, and b is the lateral dimension of the prism. The reduction from the formula for elastic flexure, $f_t = \frac{6M}{b^3}$, allows for plastic deformation. This reduction was determined experimentally by Feret.

The two halves of the broken test specimen are placed in a compression machine (one on top of the other so that the area of contact is b^2), and tested to destruction. The unit compressive strength is taken to be $\frac{N}{b^2}$, where N is the total load at failure. Tests are made at two days, seven days, and twenty-eight days, and specimens are set aside for tests at ninety days and one year. The apparatus required for these field tests was devised by M. Feret and is shown in Fig. 1.

The "characteristic" of the concrete (designated as Y_2), which is used as a basis for determining working stresses, is the unit compres-

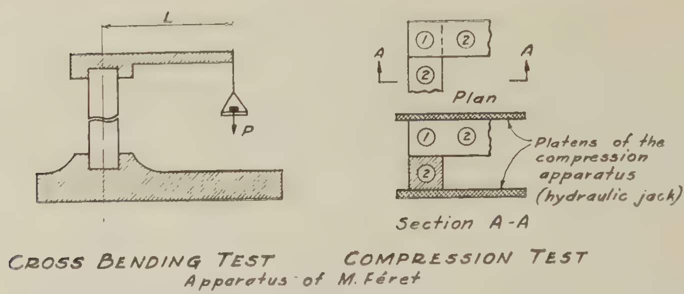

FIG. 1

sive strength at 28 days (determined as indicated above), plus the increase in compressive strength up to the time when the structure is placed in service. However, this increase is limited to 20 per cent and the amount of the increase must be determined by test for each cement. Y_2 is expressed in "hectopiezes"—a hectopieze being 1.02 Kg/cm^2. The working stress is a fixed percentage of the "characteristic" thus determined.

Nothing is said about the method of storing test specimens and it is assumed that they would be cured and stored under job conditions.

The regulations prescribe that for usual projects the "characteristic" of the concrete will be between 160 and 250 hectopiezes, corresponding to strengths of 2300 to 3600 p.s.i. They recommend:

for ordinary structures characteristic 160 (2300 lb. concrete), for structures requiring more care characteristic 200 (2900 lb. concrete) and for structures requiring very great care characteristic 250 (3600 lb. concrete).

The corresponding maximum working stresses for concrete in compression are 830, 1040, and 1300 p.s.i.

Concretes of characteristic greater than 250 (3600 lb. concrete) should only be used for special works of the highest class which are constructed by concrete specialists. They should be used only in cases of absolute mecessity and after numerous tests of adequate duration.

The commentary indicates the following approximate cement factors for concrete made with portland cement meeting our A. S. T. M. specifications, and having 1 inch aggregate, good plasticity and strengths averaging 20 per cent in excess of the standard requirements noted above.

	Cement factor—sacks per cubic yard of concrete
2300 lb. concrete	5.18*
2900 lb. concrete	5.02
3600 lb. concrete	6.00

*This factor is for cement of lower strength than our standard. No factor is given for this mixture with cement meeting our specifications, it being apparently contemplated that the higher quality cement will not be used for this class of concrete.

The cement factor in each case varies inversely as the fourth root of the maximum size of aggregate. It is stated that these cement factors must be increased in the case of wetter concretes, aggregates which are not so well graded, and where the structures are to be placed in service at an early age. Also, that these cement factors are given only as a guide; the mixture should be based on the effective strengths determined at the site rather than upon the standard cement factors.

The report states:

> The concrete may be subjected to elastic stress in any direction. When the stress on any element reaches the limiting value determined by the attached curve, called "Curve of intrinsic resistance," there will be failure.

The "curve of intrinsic resistance" is the envelope of the Mohr's circles drawn for the principal stresses at failure. The curve was determined experimentally for an average concrete. Data are given to enable the engineer to determine this curve for any concrete. The general equation of the curve is $\nu + \theta = \tau^{3/2}$.

where ν = normal component of the stress
τ = tangential component of the stress
θ = tensile strength of the concrete.

The curve is based on the assumption that concrete under combined stresses behaves like an homogeneous and isotropic material. The propriety of this assumption has been questioned by other investigators. (Ref: "A study of the failure of concrete under combined compressive stresses," by Frank E. Richart, Anton Brandtzaeg, and Rex L. Brown, Bulletin No. 185, Engineering Experiment Station, University of Illinois). The regulations also include a "curve of principal stresses" which shows the relationship between the maximum principal stress and the minimum principal stress at failure. This curve can be constructed from the "curve of intrinsic resistance." Fig. 2 shows the "curve of intrinsic resistance."

Consideration of the effect of combined stresses is a refinement not generally encountered in American practice.

Design

Calculation of Strength

> The determination of the strength of a structure must take into account dead and live loads, temperature effects, shrinkage stresses, and stresses during construction. Loading tests on typical members are accepted as conclusive proof of strength.

The tensile strength of concrete is taken into account only in computing deformations, except in slabs and beams where diagonal tensile stress exists as a result of the shearing forces.

The dead weight of reinforced concrete to be used in the design is

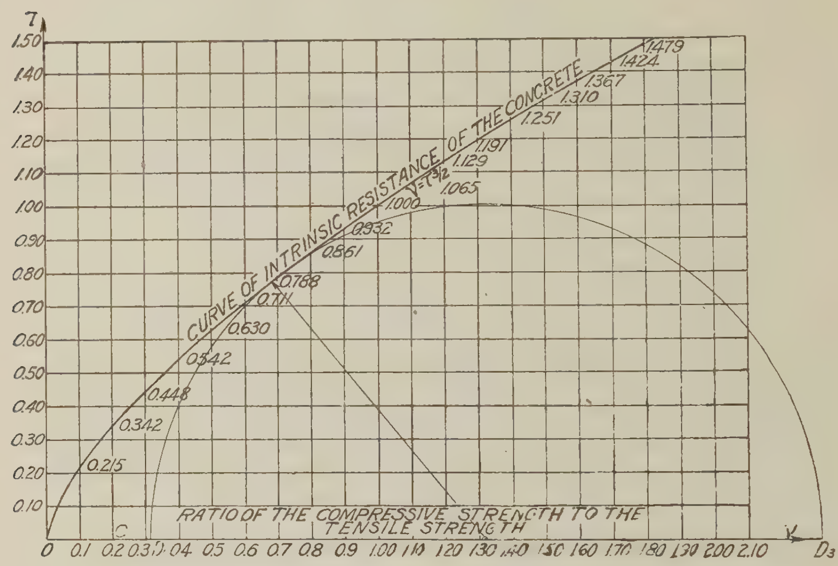

FIG. 2

made to vary with the percentage of reinforcement; for percentages between 1.0 and 2.5 an average value of 156 pounds per cubic foot is recommended.

The modulus of elasticity of steel is taken to be 32,000,000 p.s.i. For concrete, in the absence of specific experimental determinations, it is assumed that the modulus varies with the degree of humidity and the duration of loading, as follows:

(a) in water or in saturated air, $E_c = 1{,}450{,}000$ p.s.i.

(b) Under normal atmospheric conditions, for permanent loads $E_c = 1{,}450{,}000$ p.s.i. and for instantaneous loads $E_c = 4{,}350{,}000$ p.s.i.

For concretes of alumina cements these values are increased by 50 per cent.

To simplify computations, an average value of

$E_c = 2{,}900{,}000$ p.s.i. is used except in calculations to determine buckling stresses.

Poisson's ratio is assumed to be 0.15.

It is interesting to note in the foregoing several significant differences from American practice, as follows:

(a) The modulus of elasticity of steel is higher than the figure we use, (30,000,000 p.s.i.)

(b) No variation in E_c is made for variations in the quality of the concrete.

(c) The modulus of elasticity of a concrete in saturated air and under any condition of loading, is indicated to be the same as that of a dry concrete under sustained loading. This would indicate small plastic flow of the wet concrete, which agrees with the results of tests by the Institute's "Column Committee," but those tests indicate a much larger modulus for saturated concrete than for dry concrete under sustained loading.

Shrinkage and Temperature Changes.

Concrete subjected to atmospheric exposure should, except when specific experiments indicate otherwise, be considered as subjected to linear variations as follows:

1. An average shrinkage of 0.0002 in all regions except south of the line from Bayonne to Grenoble, where the shrinkage will be taken as 0.0003.

2. A variation of oscillatory character with effect opposite to that of the slow variations of temperature noted hereinafter. (This is the change of humidity effect).

3. The effect of slow variations of temperature between winter and summer, with a maximum of ± 12 degrees centigrade.

4. The effect of rapid variations of temperature of ± 12 degrees centigrade.

For the determination of stresses in the *concrete,* account is taken only of (a) the average shrinkage with $E_c = 1{,}450{,}000$ p.s.i., and (b) the rapid variation of temperature with a coefficient of expansion of 0.000006 per degree F. and $E_c = 2{,}900{,}000$ p.s.i. It is assumed that the oscillatory effects (item 2 above) are cancelled by the seasonal variations in temperature (item 3 above).

For the *steel,* account must be taken of all causes of variation in linear dimensions.

For concrete in contact with the ground or in a saturated atmosphere, shrinkage effects are neglected.

For concrete in contact with a sheltered and heated atmosphere, the shrinkage factor is 0.0006, with $E_c = 1{,}450{,}000$ p.s.i.

The foregoing recommendations were adopted as a result of Freyssinet's experiments in connection with the construction of the Plougastel bridge from which he evolved his "corpuscular theory" of cements. A brief review of this theory is given in the November-December 1933 issue of the JOURNAL of the Institute.

Design Loads.

All live loads of appreciably varying intensity are increased by 33⅓ per cent to allow for the difference in strength of a member subjected to sustained static load and the "endurance limit" of the same member under varying stresses, which are not necessarily completely reversed. In addition, impact allowances are made for moving loads. These are determined by the formula used for steel bridges. An impact allowance of 100 per cent is made for dance floors and assembly halls.

Allowable stresses.

The maximum allowable stress in the concrete is 32 per cent of the "characteristic" determined as described hereinbefore for the dead load and live load of small variations; and 36 per cent of the "characteristic" for all of the loads combined, (including impact, shrinkage, temperature, wind, snow, etc., variable live loads being multiplied by the factor 4/3). The strength of the concrete in the direction of the stress is determined from the curve of "intrinsic resistance" mentioned hereinbefore. No distinction is made between the allowable concrete stress in flexure and that in direct compression.

For the steel, direct stress only is considered, shearing stresses being neglected except in built up members used for reinforcement. The allowable stresses are 53 per cent and 60 per cent of the yield point stress for the same loading conditions indicated above for the concrete.

The concrete working stresses are lower than ours for flexural stress but much higher for column stresses. The steel stresses are higher than ours.

Compression steel.

The stress in compression steel is calculated by taking account of the linear variations in the concrete (shrinkage, temperature, etc.) and the variations in the modulus of elasticity of the concrete noted hereinbefore.

Buckling of the bars is prevented by ties spaced not further apart than twelve bar diameters. A special caution is given in regard to effective splicing of compression steel. Hooked ends are considered useless for this purpose. Electric welding is very much favored.

Tension steel.

The value of n to be used for finding the position of the neutral axis is not the ratio of the moduli, but varies with the percentage of the steel based on the gross area of the concrete. The formula for rectangular members is

$$n = 10 + \frac{bh}{20A}$$

where, b = width of member at the steel
h = total height of member
A = area of tensile steel

For $\frac{A}{bh}$ = 1 per cent, n = 15

It should be noted that no consideration is given to the effect on n of changes in the quality of the concrete.

To permit the preparation of tables, a variation of 20 per cent from the value of n indicated by the formula is permitted.

For members having irregular cross-sections and members subjected to bending and direct stress, the position of the neutral axis must be determined using the values of E_c given hereinbefore for the specific loading conditions.

Buckling.

Members subjected to compressive stress must be investigated for buckling, using the generalized formula of Rankine. The formula is:

$$\frac{N}{A}\left((1 + \frac{Kl^2}{10{,}000r^2}\right) + \frac{Mv}{I} < f_c$$

where, N = Normal thrust on the section midway between supports.—Kg.
A = transformed area of section—cm^2
l = length of member—cm
r = least radius of gyration—cm^2
$\frac{Mv}{I}$ = flexural fibre stress due to moment M at critical section—Kg/cm^2
f_c = allowable fibre stress on concrete, Kg/cm^2

K is a coefficient varying from $\frac{1}{4}$ to 4, depending upon the end conditions of the member.

It is stated that curved members shall be analyzed using "their special formulas."

Bond.

The ultimate bond strength is a function of the "characteristic," or strength of the concrete. This function varies with the ratio between the diameter of the bar and the minimum distance from the axis of the bar to the two closest free surfaces at right angles to each other. The formula is:

$$C = \frac{0.2}{\left[\left(1 + \frac{d_1}{e_1}\right)\left(1 + \frac{d_1}{e_2}\right)\right]}$$

where, C = ratio of ultimate bond strength to the "characteristic" of the concrete.
d_1 = diameter of bar (mm).
e_1, e_2 = minimum distances from the axis of the bar to the free surfaces of the concrete in two directions at right angles to each other, (m.m.)

When stirrups are used, the distances e_1 and e_2 are increased by a fictitious thickness (in centimeters) equal to the cross section (in square centimeters) of the stirrups encountered in a length of one meter.

The working stresses are 32 per cent and 36 per cent of the ultimate strength, under the conditions of loading described hereinbefore.

Mesnager states that the bond resistance can be doubled by the effective use of stirrups.

Diagonal tension reinforcement.

The treatment is, in general, similar to American practice, except that where the diagonal tension exceeds the safe shearing allowance for concrete, the shearing reinforcement is designed to take the entire stress, no allowance being made for the concrete.

Mesnager states that vertical stirrups do not prevent rupture of the concrete by shearing forces; they serve only to prevent opening of the cracks after they have formed. Torsion tests on cylinders indicated that the concrete cracks under the same load regardless of the character of the reinforcement.

He states that stirrups inclined at 45 degrees prevent the formation of cracks under working loads, by reason of the "stretching" of the concrete under constant load. However, the ultimate strength of the member is the same for vertical and inclined stirrups.

Vertical stirrups are used almost exclusively on account of the ease of fabrication and handling.

The maximum allowable working stress in shear for concrete is 36 per cent of the ultimate fibre stress in tension determined by test as indicated above. In the absence of specific information, the shearing resistance is assumed to be 1/12 of the compressive strength.

Lateral Reinforcement for Compression Members.

The increase in strength resulting from lateral reinforcement is determined by a formula which has as variables the form of the lateral reinforcement (circular bands, square bands, rectangular bands, etc.), the percentage of transverse reinforcement, spacing of the transverse reinforcement, the smallest lateral dimension of the concrete section, and the yield point of the lateral steel. The formula is:

$$P = A\left[Y_2 + \lambda\alpha\left(1 - 2\frac{e_4}{b}\right) Y_1 \times 180\right]$$

where, P = total supporting power of the member, Kg.

A = gross area of the member, including the equivalent area of the longitudinal steel, cm^2

α = volumetric percentage of lateral reinforcement based on the gross area of the member, and excluding laps in the steel.

e_4 = distance center to center of lateral reinforcement bands, cm.

b = smallest lateral dimension of the concrete, cm.

Y_1 = "characteristic" (yield point) of the lateral steel, myriapiezes (a myriapieze is 1.02 Kg. per mm^2).

Y_2 = "characteristic" of the concrete, hectopiezes.

λ = a factor dependent upon the disposition of the lateral reinforcement. For circular bands $\lambda = 2$, for square bands $\lambda = 1$, and for rectangular bands $\lambda = b/a$, where a is the largest lateral dimension of the concrete section.

The regulations state that "when there is a possibility of premature separation of the shell outside the lateral reinforcement, the allowable supporting power shall be not greater than $2AY_2$."

When the unit stress does not exceed $0.8Y_2$, the value of E_c is determined as specified hereinbefore, according, to the method of loading. For stresses in excess of $0.8Y_3$, the value of E_c should be determined experimentally.

Certain features of this treatment are of special interest in view of the recent extensive research on this subject which was sponsored by the Institute. These are as follows:

(a) Allowance is made for the concrete outside the lateral reinforcement in determining the ultimate strength of the member.

(b) The ratio of effectiveness of spiral lateral reinforcement to that of an equal amount of vertical reinforcement, assumed by the Institute's Committee to be 2, is assumed to vary from a theoretical maximum of 3.6 (when $2e_4/b = 0$) to a minimum of zero (when $2e_4/b = 1$). Therefore, lateral bands spaced farther apart than one-half of the least lateral dimension of the member are considered to be ineffective. For all closer spacings, an allowance for increase in strength is made. For an average case ($e_4/b = 1/5$, $\lambda = 2$, $\alpha = .015$), the effectiveness ratio would be 2.2. The minimum spacing of lateral reinforcement is governed by the size of the aggregate used.

(c) The Institute's Column Committee proposes to make no allowance for other than circular spitals. The French Commission would permit rectangular lateral bands, with smaller effectiveness than circular bands.

(d) The French do not limit the amounts or spacings of lateral reinforcement except as the spacing is governed by the size of the aggregate.

(e) There is no limitation on the *increase* in strength which can be obtained by the use of lateral reinforcement except that "when there is a possibility of premature separation of the shell," the increase is limited to 100%. The majority recommendation of the Institute's Column Committee limits the increase to 25 per cent, which results from the use of a factor of safety 25 per cent higher for columns with lateral ties.

(f) The effectiveness of the longitudinal reinforcement is based upon the ratio of the moduli of elasticity, but the modulus for concrete varies with the degree of humidity and the duration of load. For high working stresses, the Commission recommends experimental determination of the modulus for concrete.

Some of these differences in treatment probably arise from the fact that the French regulations are intended to cover all compression members, while the Committee of the Institute was concerned only with a particular type of compression member, namely, building columns.

It is interesting to note the following statement in the Commission's commentary:

Empirical formulas should be limited as much as possible, and in the present regulations we have introduced only those which were absolutely necessary to determine the data in question. The commission has carefully avoided approximate formulas with respect to bending moments, shearing stresses, etc. These approximate formulas rapidly become, in practice, dogmatic rules and frequently result in serious consequences in actual construction . . . It is thus that the formula, $M = \frac{pl^2}{10}$ can

be considered only as a first approximation, but should not be used to determine the final dimensions of the structure.

It is noted that in most cases the recommended formulas do not lend themselves readily to the compilation of tables. French engineers are opposed to excessive standardization of design procedures. They believe that voluminous and detailed charts, tables, instructions, and regulations have a stifling effect on originality and initiative. The French engineer enjoys a very high standing in the community. Perhaps this may be attributed in some measure to his reluctance to simplify his professional equipment to an extent such that engineering tools are made available to those who should not be entrusted with their use.

For such discussion of this paper as may develop, readers are referred to the JOURNAL *for November-December 1934 (Proceedings Vol. 31). Discussions should be available to the Secretary by October 1, 1934.*

Concrete as a Medium of Architectural Expression in Building*

BY ALFRED CHAPMAN†

In the convention presentation, Mr. Chapman's brief address was introductory to stereopticon views, some of which are here reproduced. His comments on each picture are here published as captions for the illustrations which for convenience in publication are sprinkled through the text.—EDITOR

I should like to make it clear that I am not before you as any kind of authority on the use of concrete as an architectural medium, but I am simply going to try to give you an architect's point of view.

Architects may seem to lack enterprise, but if you consider their position you will see that there is reason for this. An architect is responsible for his client's investment, and it is often a large investment in proportion to his means. Consequently security is sought; an avoidance of leading his client over untrodden paths even though these may lead to an alluring adventure for the architect personally.

A painter, musician or author may explore unknown fields with impunity knowing that if unsuccessful he alone suffers. No so an architect, as his failure means a loss to those who have trusted him. A building has to stand in a public place and face condemnation or approval, and even though the judgment of the architect may be vindicated in a decade or so, it is no consolation for the client in the meantime.

Personally, I feel that concrete as a means of architectural expression has great possibilities. The essential principle of the constructions stirs the imagination. The Egyptians built with huge blocks of granite—the Grecians in blocks of marble—the Gothic arches curved aloft composed of comparatively small stones set cunningly to obtain equilibrium, but you can build any shape, size or strength into a fantastic monolith: a monolith as strong as stone in compression

*Presented at the 30th Annual Convention, American Concrete Institute, Toronto, Feb. 20-22, 1934.
†Chapman & Oxley, Architects, Toronto.

FIG. 1—ROGUE RIVER BRIDGE, OREGON, DESIGNED ON THE FREYSSINET SYSTEM

In this bridge we can appreciate the extent of the achievement in monolithic concrete construction. The form dictated by constructional requirements produces a pleasing effect of lightness and airiness not usually associated with concrete construction.

and as strong as steel in tension. This is the wonder of it and the element that should find expression.

The duty of the architect is to grasp this new principle and interpret it.

A building should first express the practical function it serves, then be beautified by conforming to the laws of proportion in mass and line, but it should also, if possible, be an honest expression of its means of production. The feeling of sincerity is deeply embedded in human nature. We like a thing to be consistently, throughout, what it appears to be on the surface, and this is an important principle of architecture.

A steel or concrete frame filled with brick or tile and faced with stone that looks solid and as though it were doing most of the work, but in reality is just a thin veneer, is an architectural lie that many of us are guilty of. A concrete wall with the face dressed and possibly the junction of pours that shows the different operations featured as

FIG. 2—WALNUT LANE BRIDGE, FAIRMONT PARK, PHILADELPHIA

This bridge was built in 1908. Except for the floor system, the entire structure is of un-reinforced concrete. The arch is 233 feet in length and 150 feet high. This makes a noble structure of a heavier character than the preceding one. The indication of arch stones is a concession to the traditional masonry construction which detracts from the expression of its true method of construction. Concrete construction, when this was built was much younger and lacked the self-assurance it is today assuming of coming boldly forth and standing on its own merits.

one of the architectural elements composing the design would be a sincere expression of construction.

Now let us face the important architectural question. Can a surface for structural concrete be developed that in color, texture and weathering propensities would compare favorably in its naked simplicity with natural stone? By this I do not mean that it must imitate stone, but would have its own character of texture as it has in its method of construction. There is no doubt in my mind that this is physically possible and it seems to me that it should be possible economically.

FIG. 3—SEABOARD NATIONAL BANK, LOS ANGELES

This and Fig 4 give two examples of concrete construction used for small commercial buildings.

Most of the methods of treating the surface come under the following heads:

Leaving the concrete frankly as it comes from the forms.
Bush hammering the surface.
Rubbing the surface.
Acid washing to clean the aggregate.
Painting.

And then we have the great variety of surfaces used in pre-cast work, but this latter work as far as architectural design goes is just another type of stone work, and does not express the particular character of concrete construction.

It is, however, in some of the textures produced in pre-cast work that we can see the great possibilities of surface treatment, particularly those where the concrete is ground and rubbed giving a surface

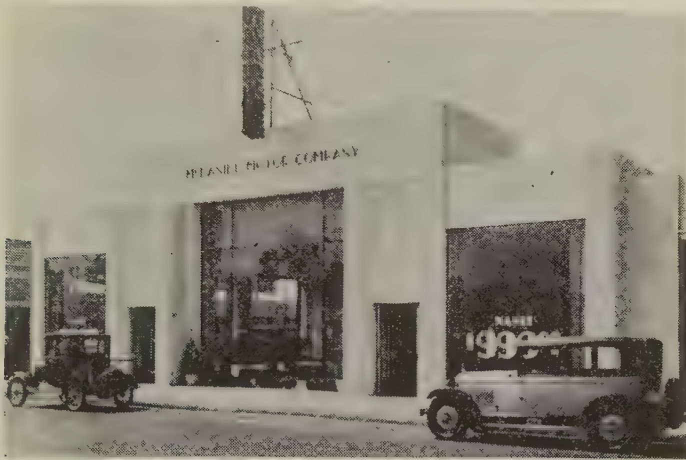

FIG. 4—McDANIEL MOTOR COMPANY BUILDING, LOS ANGELES

In this we begin to see architectural expression of the system of construction. Here is a generous monolithic mass that could hardly be done in any other system of construction. Of course, we could do all kinds of tricks inside and cover it up with plaster.

FIG. 5—WARNER BROS. THEATRE, LONG BEACH, CALIFORNIA

Here we come to a larger type of building where the solution has resolved itself into heavy blocks of massing a type that gives great scope for the application of the principles of monolithic construction. For economic reasons in a building of this type an expensive facing material cannot be used throughout, and we have in consequence that cheap result of a Queen Anne front and Mary Ann back. Here it has been avoided and the result gains in dignity.

FIG. 6—SUNSET TOWER, LOS ANGELES, CALIFORNIA, LELAND BRYANT, ARCHITECT

Here we again see a breadth of treatment that is an expression of the system of construction. The ornament where used is of an all over pattern. It is a decoration on a monolithic surface with no tendency to give an ornamental portion a separate entity with a frame or base and cornice. Also note the colossal size of the ornamental parts.

that weathers well and shows clearly the actual composition of the material and can be made very attractive. I have seen some very satisfactory surfaces produced in this manner with ordinary gravel as the aggregate.

We grind terrazzo and concrete floors by the acre. Surely this can be done on a vertical surface by suspending a grinding machine with an inclined hanger that would produce a positive pressure against the wall. If by this method, results can be produced that compare favorably with the surfacing used now in first class buildings, there is a margin of from 40¢ to $1.00 per square foot to work on besides the saving in space of from four to six inches in the whole perimeter.

It might be of interest to you if I quoted from an address made

recently by the President of the Royal Institute of British Architects in which he says:

> Even if concrete were a suitable material for large surfaces, it has in its natural state no beauty either of color or texture, and it weathers badly, getting uglier and uglier instead of more beautiful. Chiselling or bush hammering, combined with a good colored aggregate, greatly improves it, but as a material for large wall areas, it is unsatisfactory both from a practical and aesthetic point of view.

FIG. 7—DOMINGUEZ-WILTSHIRE BUILDING, LOS ANGELES; MORGAN, WALLS AND CLEMENTS, ARCHITECTS

This building does not express a distinctive type of concrete construction except, of course, its texture and the suggestion of horizontal jointing. I do not know whether these joints occur at the pouring junctions. If they do, they tell the story of the series of operations by which the building was constructed, and also serve to cover up the ragged appearance that develops at this junction. If these lines do not occur at these actual junctions, they at least serve to suggest the principle of the series of operations by which the building is constructed.

This building expresses steel construction on account of the effect of detached vertical elements whereas if a true concrete expression was sought, a series of huge monolithic bands running around the whole building would be a more suggestive method of expression.

FIG. 8—NORTON MEMORIAL AUDITORIUM, CHAUTAUQUA, NEW YORK, OTIS F. JOHNSON, ARCHITECT

All the ornament was made with waste molds and cast in the same concrete as the structural parts of the building. The cost of this building was, I believe, 23¢ per cubic foot, which forms quite an achievement in economics. I do not feel that it has a particularly monolithic feeling, as what expression is imparted by the broad simple frieze is counteracted by the framing of the rest of the design.

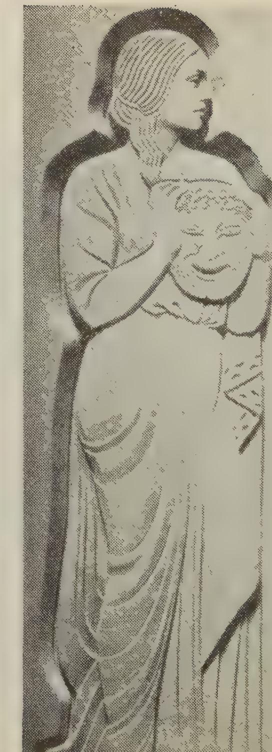

FIG. 9—DETAIL OF SCULPTURE—NORTON MEMORIAL

This shows the simple treatment of some of the modeling on the building just shown. It was done with waste molds and shows how the relief was obtained without projection beyond the face of the wall, and I think it is particularly adaptable to monolithic construction.

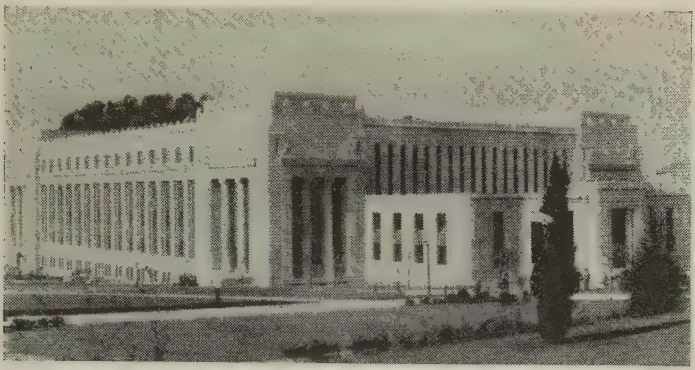

FIG. 10—LIFE SCIENCE BUILDING, UNIVERSITY OF CALIFORNIA; GEORGE W. KELHAM, ARCHITECT

Here we have a building of a kind of Neo-Grec character. Like most of traditional styles, it is not a very satisfactory type of architecture to express in concrete. The reason for this is not far to seek as the styles were developed for and by another system of construction.

I think Sir Gilbert Scott expresses the feeling many architects have for concrete as a finished surface, and certainly few would question it as applied to traditional architecture.

But we have been building with brick and stone for thousands of years, and developing various architectural expressions with this medium. Concrete construction is hardly a matter of decades.

The development of commercial architecture is a matter of only a few decades and in this class of work I believe that concrete construction has its greatest future.

The feeling of building in large monolithic masses is against all tradition, but is the abstract principle any the less sound for this reason? It is hard for us to estimate to what extent we are enslaved by tradition, but let us break the bounds in our imagaination and suppose we had been building in large monolithic masses for hundreds of years with no knowledge of brick. If some enterprising person came along and suggested that we should cover our buildings with little burnt clay blocks, I think we would feel that the idea was so laborious and finicky that we would not entertain it.

If somebody suggested a covering of stone, the idea would be more comprehensible to these imaginary builders of monolithic structures and the acceptance of the idea would depend on the comparative

(Address concluded on page 420)

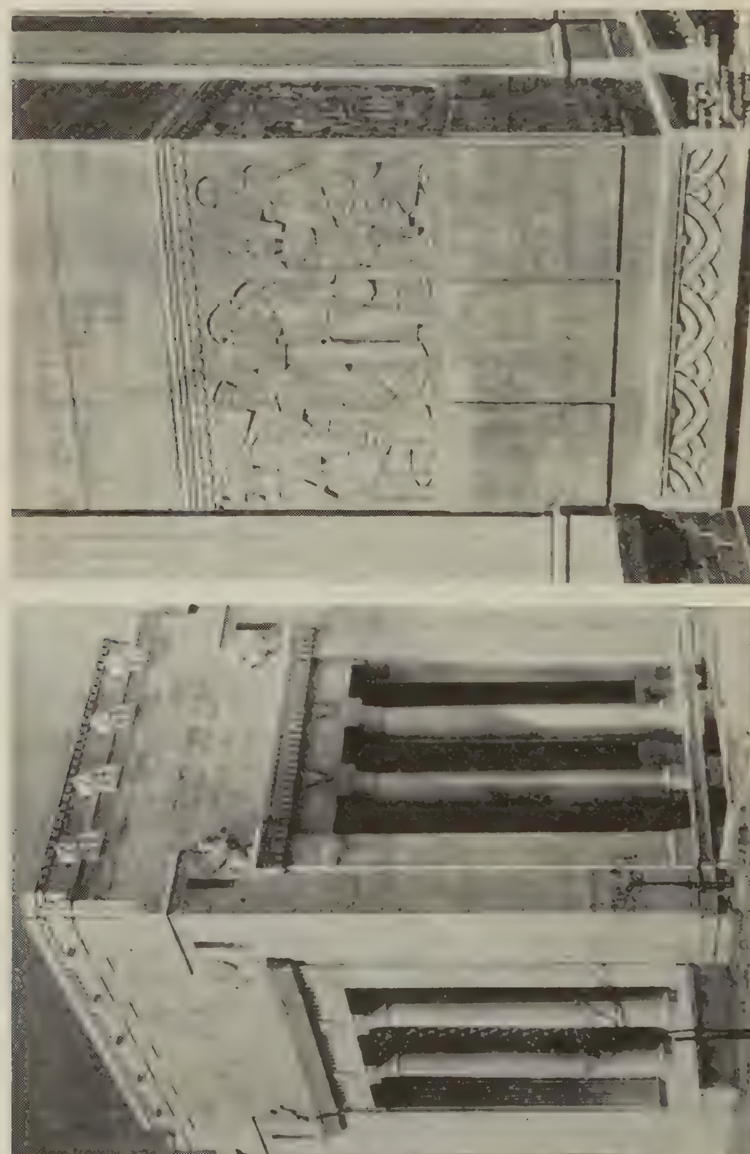

FIG. 11 AND 12 (SEE TOP OPPOSITE PAGE)

FIG. 11 (OPPOSITE PAGE)—DETAIL OF LIFE SCIENCE BUILDING

If we look at the corner pavillion of this building, we obtain an impression of a rectangular monolithic mass with the openings framed with architectural detail. This part of the design lends itself quite satisfactorily to expression in monolithic concrete construction. Note this modeled work of which I am going to show you a detail.

FIG. 12 (OPPOSITE PAGE)—DETAIL OF ORNAMENT

Here we have an illustration of some modelled ornament which incidentally is not Neo-Grec. This work was done by plaster waste molds set in the form work. The same principle of obtaining relief was used as shown on a previous illustration of modelling. This illustration is interesting as it shows clearly the texture left by the removal of the forms. It is not a beautiful texture, nor would it weather well, but it is an honest surface—"a poor thing but mine own."

I feel that the roughness left by the forms, certainly as seen from any distance, is preferable to having the walls rubbed down to a dead monotony.

FIG. 13—ST. JOSEPH'S CHURCH, SEATTLE, WASHINGTON; A. H. ALBERTSON, ARCHITECT

The truth in the expression of construction that is particularly desirable in Church building together with the large masses of masonry that develop from the nature of the problem, give an interesting field for the use of monolithic concrete construction.

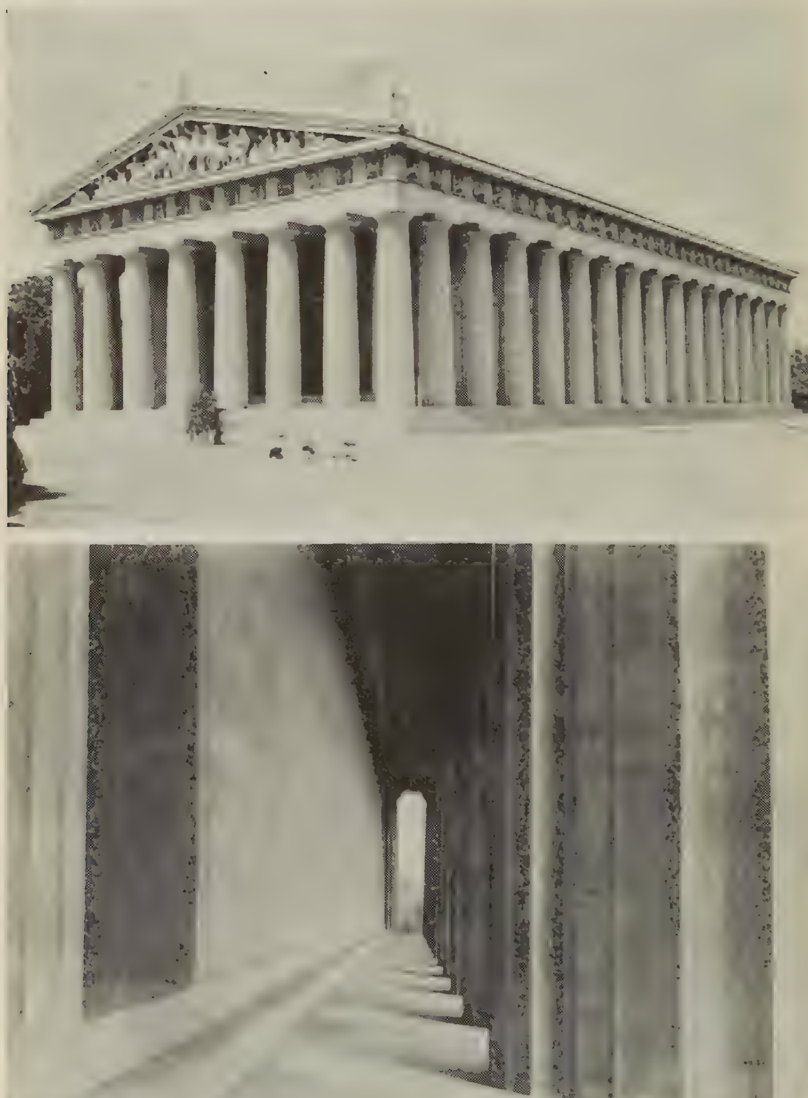

FIG. 14 AND 15 (SEE TOP OPPOSITE PAGE)

FIG. 14 (OPPOSITE PAGE)—THE PARTHENON, NASHVILLE, TENNESSEE, GEORGE O. NEVINE AND RUSSELL E. HART, ARCHITECTS. A REPLICA OF THE PARTHENON AT ATHENS BUILT AND USED AS AN ART MUSEUM. THE CONCRETE WORK, BOTH INSIDE AND OUT WAS DONE BY JOHN J. EARLEY.

The general color of the structure is buff, while the background of the pediment and the metopes are produced in the colors of the original building.

Here we have an achievement, the interest of which I imagine lies, from the concrete construction point of view, in the effect of the texture and the ability with which the concrete has been handled. It is however, the reproduction of a marble building wonderful in its perfection of design and workmanship, and the cost of reproducing it in any other material than concrete would be very much greater.

FIG. 15 (OPPOSITE PAGE)—DETAIL OF NASHVILLE PARTHENON

Here is an interesting photograph of the peristyle of the building previously shown. This gives an idea of the texture and the perfection of workmanship. I am hoping we shall learn more about this building from the subsequent papers. This wall is particularly interesting. At present I do not know whether these are monolithic pours with the texture varied or whether it is composed of pre-cast blocks. If the former, it opens up an interesting possibility of getting away from the dead monotony of a wall of monolithic concrete construction. (Mr. Earley answered this question—see March-April JOURNAL, this volume, p. 277.—EDITOR)

FIG. 16—LOS ANGELES COUNTY GENERAL HOSPITAL, LOS ANGELES; DESIGNED BY AN ASSOCIATED GROUP OF ARCHITECTS

This building has a structural steel frame and an exterior of monolithic concrete. Here we have a very large building which from its size and massing would carry off the crudity of surface produced by leaving ordinary concrete frankly as it comes from the forms. It is a courageous, sincere achievement and the economy effected, so important in this type of building, disarms the detail criticism of the surface texture. This is a big step in advance for monolithic concrete in a large building or group of buildings particularly where economy is an important element.

FIG. 17—BOWLES DORMITORY, UNIVERSITY OF CALIFORNIA; GEORGE W. KELHAM, ARCHITECT

Here we have a traditional type of architecture; the association of which is bound up in stone. It does not express concrete construction which to me seems a very unsatisfactory substitute for the stone which is part and parcel of this particular style, and is responsible for a great deal of its charm.

beauty of the surface they had developed with that of the stone, and whether it would compensate for the more laborious and complicated principle of construction.

I also think these imaginary monolithic builders would have developed a breadth of treatment beautified by molded form that they would be loath to part with.

After this somewhat lengthy preamble, we shall now turn to some slides that will illustrate some of the progress that is being made in concrete construction. I would like to make it clear that, in my remarks about the buildings, views of which are to be shown, I am not pretending to deal with the architectural merits or demerits of the designs as a whole but only as they appear to me in their particular expression of concrete construction. (The author's comments on the pictures are presented in the captions of the accompanying illustrations. His closing remarks were:—EDITOR)

I think you will agree that the illustrations shown indicate that very considerable progress has been made in this comparatively

new medium for architectural expression, and give an encouraging indication of what progress will probably be made in the future.

The modernistic tendency to simplify traditional architectural form and to accentuate the expression of function and construction gives a wonderful field for the development of monolithic concrete construction as a new medium for architectural expression.

FIG. 18—MANIOR RICHELIEU, MURRAY BAY, QUEBEC; JOHN ARCHIBALD, ARCHITECT.

The original hotel was destroyed by fire in September, 1929. This new, modern, fireproof structure done in monolithic concrete was ready for occupancy in June of the following year. The speed necessary in construction together with the economy effected and the Chateau type of design adopted for these hotels in Canada, I presume is the justification for the use of concrete for a style that is traditionally associated with stone. Like in the building previously shown a lot of the charm of this style would be lost by not having the material that it is associated with and grew out of, but on account of its isolation in rugged natural surroundings the defect does not detract from the general impression to the same extent.

For such discussion of this paper as may develop, readers are referred to the JOURNAL *for November-December 1934 (Proceedings Vol. 31). Discussions should be available to the Secretary by October 1, 1934.*

Bonding of New Concrete to Old at Horizontal Construction Joints*

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SCOPE OF TESTS

To DETERMINE the relative efficiency of various methods of bonding new concrete to old under conditions similar to those that exist along horizontal construction joints in dams, three series of tests were carried out during the last year in the Engineering Materials Laboratory at the University of California.

The specific purpose of the tests was to investigate the effect of several methods of placement and several methods of joint treatment upon (a) permeability along the plane of the joint, (b) bond strength of the joint as indicated by a flexure test, and (c) quality of the concrete as indicated by compressive strength, unit weight, and absorption.

The methods of compaction of the concrete were (a) hand tamping, (b) internal vibratory tamping and (c) a combination of internal and surface vibratory tamping.

The methods of cleaning the surface of the lower lift were (a) wire-brushing shortly after final set had taken place, (b) scouring with a high-pressure air-water jet between the time of initial and final set, so that the cement paste was washed from the surface aggregate, and (c) cleaning with a high-pressure air-water jet after final set had taken place, so as merely to remove any surface scum or laitance. In addition to these cleaning operations, for some of the specimens the surfaces of the lower lifts were flushed with mortar just before the placement of the concrete of the upper lift; and for the remainder of the specimens, the concrete of the upper lift was deposited directly on the surface of the lower lift. Also, for specimens employing the wire-brush method of cleaning, the surfaces of the lower lifts were washed with air and water before flushing with mortar.

The effect of the following variables upon bond strength and permeability was studied: (a) type of cement, (b) richness of mix, (c)

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consistency of the fresh concrete as regulated by the water content of the mix, (d) period of exposure of the surface of the lower lift to drying conditions similar to those which exist in arid regions during the hot summer months, (e) time interval between lifts, and (f) age of concrete at time of test.

DETAILS OF MANUFACTURE AND TESTING OF SPECIMENS

Series I

The test specimens of Series I were 2½ by 2½ by 5-ft. prisms cast in two lifts, each of depth 2½ ft., and 6 by 12-in. control cylinders. The aggregates used were local gravels of maximum size 3 in.

Two cements were included, (a) a special cement of relatively high dicalcium-silicate content and low heat-generation, meeting the specifications for a low-heat cement for Boulder Dam, and (b) a standard portland cement of medium heat-generation. The cement content of the majority of the mixes was 1.2 bbl. of cement per cu. yd. for the concrete having 3-in. maximum size of aggregate, which was equivalent to 1.0 bbl. of cement per cu. yd. for a concrete having 9-in. maximum size of aggregate;* but for one sub-group, the cement content was 1.0 bbl. of cement per cu. yd., which was equivalent to 0.8 bbl. of cement for a concrete with 9-in. maximum size of aggregate. The data regarding the mixes and gradation of the aggregates are given in Tables 1A and 1B.

*Same surface area of aggregate per bbl. of cement and approximately the same water-cement ratio.

TABLE 1A—GRADATION OF AGGREGATES FOR VARIOUS MAXIMUM SIZES

Sieve No.	Per Cent Passing Given Sieve		
	9-In. Aggregate	3-In. Aggregate	1½-In. Aggregate
100	0.5	0.6	0.8
48	3.9	4.8	6.1
28	14.4	18.0	22.9
14	18.3	22.8	29.0
8	21.4	26.6	33.9
4	25.8	32.1	40.8
⅜ in.	33.4	42.1	54.1
¾ in.	44.2	56.5	73.1
1½ in.	59.6	76.8	100.0
3 in.	77.1	100.0	—
6 in.	93.1	—	—
9 in.	100.0	—	—

TABLE 1B—PROPORTIONS OF MIXES

NOTE: For the various maximum sizes of aggregate the mixes were adjusted so as to have the same surface area of aggregates per unit of cement.

Material	Relative Weights of Materials for				
	9-In. Aggregate		3-In. Aggregate		1½-In. Aggregate
Cement	1	1	1	1	1
Sand	2.45	3.06	2.45	3.06	2.45
Gravel	7.05	8.81	5.18	6.47	3.55
Cement content, bbl./cu. yd., 3 to 4-in. slump basis	1.0	0.8	1.2	1.0	1.4

A consistency corresponding to a slump of 2½ in. was used for all hand-tamped specimens, and consistencies corresponding to slumps of 1½ and ½ in. were used for the concretes compacted by vibratory tamping. The average water requirements for the various mixes are given in Table 2.

There are also included in Table 2 data of interest relative to (a) the average time required to compact similar mixes in the forms, (b) the average quality of the concrete as determined from compression tests made on 6 by 12-in. control cylinders and on corresponding 2½-ft. cubes and (c) the unit weight of the concrete and its absorption. The mortar for flushing the joints had the same cement-sand ratio as did the corresponding concrete, and its slump was 8 to 9 in. For the majority of the specimens, the upper lifts were placed 3 days after the lower lifts; for the remainder of the specimens, the interval between lifts was 12 days. All specimens were stored under moist conditions at 70° F. until time of test.

Tests made at ages of 28 days and 3 months consisted in (a) applying water under pressure of 100 lb. per sq. in. for 12 hours to the center of the joint plane of each prism through a porous block, (b) determining the bond strength at the joint by testing each prism in flexure as a simple beam with a centrally applied load on a span of 58 in., (c) crushing the 2½ ft. cubes remaining from the flexure test, and (d) determining the moisture content and absorption of the concrete.

TABLE 2—CHARACTERISTICS OF CONCRETE MIXES—SERIES I

NOTE: Values given are averages.

Kind of Cement	Cement Content, bbl./cu. yd.	Water-Cement Ratio, by wt.	Slump, in.*	Flow, per cent*	Compaction		Compressive Strength, p.s.i.				Unit Weight of Prisms, lb./cu. ft.	Absorption, % of Dry Wt.
					Method	Relative Time	Control Cyls.*		2½-Ft. Cubes			
							28 Da.	3 Mo.	28 Da.	3 Mo.		
Special Low-Heat	1.2	0.57	2½	172	Hand	100	2760	4780	2190	3760	153.2	5.6
		0.54	1½	153	Vibration	22	3190	5180	2510	4130	153.9	5.6
		0.50	½	135	Vibration	44	3300	5340	2900	4400	154.1	5.6
	1.0	0.68	2½	178	Hand	100	2100	3880	1680	3540	153.0	6.4
		0.64	1½	160	Vibration	21	2220	4020	1780	3650	153.7	5.8
		0.60	½	140	Vibration	39	2390	4270	2150	3870	154.5	7.3
Standard	1.2	0.59	2½	173	Hand	100	3890	4980	3480	4200	152.8	5.5
		0.52	½	135	Vibration	42	4150	5210	3930	4380	153.9	5.1

*Concrete wet-screened to 1½ in. maximum size of aggregate.

Series II

For Series II the test specimens and methods of test were similar to those of Series I, but the aggregates were gravels up to a maximum size of 6 in., and the cement was a blend of four brands of low-heat portlands similar to those used in the construction of Boulder Dam. The cement content of the mixes was 1.0 bbl. per cu. yd.

A consistency corresponding to a slump of 3 in. was employed for specimens compacted by hand tamping; and a consistency corresponding to a slump of 1½ in. was employed for specimens compacted by internal vibration.

The characteristics of the concrete mixes used in this series are given in Table 3.

TABLE 3—CHARACTERISTICS OF CONCRETE MIXES—SERIES II

Cement: Blended low-heat cement.
Cement Content of Mixes: 1.0 bbl. per cu. yd.

Water-Cement Ratio, by Wt.	Slump,* in.	Flow,* per cent	Compaction		28-Day Compressive Strength, p.s.i.		Unit Weight of Prisms, lb./cu. ft.
			Method	Relative Time	Control* Cylinders	2½-Ft. Cubes	
0.48	3	174	Hand	100	3790	2960	156.9
0.45	1½	147	Vibration	44	3930	4060	156.9

*Concrete wet-screened to 1½ in. maximum size of aggregate.

Series III

For Series III, the specimens were 8 by 8 by 36-in. prisms with construction joint at mid-height.

Local gravels having a maximum size of 1½ in. were used (see Table 1A). The mixes contained 1.4 bbl. of special low-heat cement per cu. yd. of concrete, which was equivalent to a cement content of 1.0 bbl. for a concrete with 9-in. maximum size of aggregate.

There were included in the series concretes of three consistencies as indicated by the slump test: (a) 2½ in. slump, compacted by hand tamping, (b) 1½-in. slump, compacted by internal vibration, and (c) ½-in. slump, compacted by internal vibration. The characteristics of the concretes are given in Table 4.

TABLE 4—CHARACTERISTICS OF CONCRETE MIXES—SERIES III

Cement: Special low-heat.
Cement Content (actual): 1.4 bbl. per cu. yd.
(equivalent): 1.0 bbl. per cu. yd.
Compression tests were made on prisms 8 by 8 by 18 in.
Values reported are averages of specimens subjected to all storage temperatures.
Compression tests at age of 28 days.

Method of Compaction	Water-Cement Ratio, by weight	Slump, in.	Flow, per cent	Compressive Strength of Prisms, p.s.i.	Unit Weight, lb./cu. ft.
Hand	0.55	2½	165	2350	150.4
Vibration	0.53	1½	147	2550	151.0
Vibration	0.49	½	130	2710	151.8

The surfaces of the lower lifts of the specimens were dried by exposure to a current of warm dry air (temperatures of 100° F. and 130° F.) for various periods up to 10 hours, beginning at the time of placement. For one group of specimens (A), the top lifts were placed on the lower lifts immediately at the end of the drying period without flushing the surfaces of the lower lifts with mortar. For a second group (B), the surfaces which were protected from drying after the end of the given drying period were cleaned with an air-water jet at 20 hours, and the upper lifts were placed at the end of 3 days, the surfaces of the lower lifts being treated with mortar immediately prior to placing the concrete of the upper lift. For a third group (C), similarly cured, the surfaces of the lower lifts were wire-brushed at the age of 12 hours and cleaned with the air-water jet. For groups B and C, the upper lifts were placed at the age of 3 days, after the surfaces of the lower lifts had been flushed with mortar.

At the end of the given periods of exposure for the specimens of group A, and after the 10-hour high-temperature period for groups B and C, the specimens were stored

in fog at 70° F. After 28 days from the time the upper lift was placed, the specimens were tested as simple beams with a centrally applied load on a 34-in. span. After rupture, the upper lift of each specimen was tested in compression, with the load applied in the direction of the 18-in. axis.

RESULTS OF TESTS; DISCUSSION

Effect of Placement Method upon Bond Strength

One of the marked results of the investigation was the difference between the bond strengths as influenced by the methods of placement. While the method of placement is defined by the means used to compact the concrete, for each method of compaction there is a limit to the dryness of the mix or to the workability which can be economically and successfully employed. It would appear, therefore, that any method such as vibratory compaction, which would permit the use of drier mixes than are possible with compaction by hand, would have, if effective, a definite advantage as the result of a reduced water-content.

The results of tests showing the combined effect of the method of compaction and consistency of mix upon the bond strength are given in Table 5. The average values of the modulus of rupture there shown are for various groups of specimens of which the surfaces of the lower lifts were flushed with mortar just prior to depositing the concrete of the upper lift, which was placed when the lower lift was 3 days old.

Consulting the table it is seen that the joint strengths of the specimens compacted by internal vibration were consistently and in many cases substantially higher than the joint strengths of specimens compacted by hand tamping. The results of the tests of Series II, given in Table 5, show a 12 per cent increase in joint strength for vibrated specimens having a 1½-in. slump as compared with hand tamped specimens having a 3-in. slump; while for the tests of Series III, also given in the table, an increase of 30 to 40 per cent was obtained. Even higher percentages of increase in joint strength of vibrated specimens are obtained for vibrated concrete of ½-in. slump.

In Series I, it is of particular interest to note that as much as 100 per cent increase in strength was obtained for vibrated specimens having the surfaces of the lower lifts cleaned with the air-water jet, over hand-tamped specimens for which the surfaces of the lower lifts were cleaned by wire-brushing. This latter method has been standard job practice over a period of years.

Effect of Method of Joint Preparation upon Bond Strength

The influence of the various methods of preparation of the joint upon the bond strength of the joint was determined by the following

TABLE 5—EFFECT OF PLACEMENT METHOD UPON BOND STRENGTH

NOTES: Joints treated with mortar.
Interval between lifts 3 days.
In Series II, a slump of 3 in. was used instead of 2½ in. as listed in the column heading.
In Series III, the values given are averages of specimens having similar conditions of storage. The surfaces of these specimens were not subjected to drying.

Series	Equivalent Mix, bbl. per cu. yd.	Type of Cement	Method of Cleaning Joint	Modulus of Rupture, p.s.i.					
				Age at Test: 28 Days			Age at Test: 3 Months		
				Hand Tamped	Internally Vibrated		Hand Tamped	Internally Vibrated	
				2½" Slump	1½" Slump	½" Slump	2½" Slump	1½" Slump	½" Slump
I	1.0	Special low-heat	Wire brush at 12 hr.	102	...	...	199	...	...
			Air-water jet at 20 hr.	...	174	189	...	217	269
			Air-water jet at 6 hr.	365	368	...	565	576	...
		Standard portland	Wire brush at 9 hr.	166	...	...	241	...	...
			Air-water jet at 15 hr.	...	...	257	...	...	298
	0.8	Special low-heat	Wire brush at 12 hr.	80	...	...	200	...	...
			Air-water jet at 20 hr.	...	144	166	...	268	277
II	1.0	Blended low-heat	Air-water jet at 6 hr.	462	516	...	...	...	...
III	1.0	Special low-heat	Wire brush at 12 hr.	283	404	412	...	...	...
			Air-water jet at 20 hr.	340	445	474	...	...	...

studies: (1) the effect of method of cleaning the surface of the lower lift, (2) the effect of the age of the lower lift at the time of cleaning with the air-water jet, (3) the effect of using a mortar layer in making the joint, and (4) the effect of using a surface vibratory tamper in addition to the internal vibratory tamper for the purpose of embedding completely large pieces of aggregate which would otherwise protrude from the mass.

In Table 6 are given data comparing bond strengths of specimens of which the surfaces of the lower lifts had been wire-brushed, with strengths of specimens using the air-water jet for cleaning the surfaces of the lower lifts. It is apparent that the air-water method of cleaning is distinctly superior to the wire-brush method.

In Table 7 are shown the results of tests made on specimens of which the surfaces of the lower lifts were cleaned at ages from 6 to 20 hours after casting. In every case, for the conditions under which

these tests were made, cleaning at an age of 6 hours resulted in substantially greater bond strengths than cleaning at any of the later ages.

When joint surfaces were cleaned with the air-water jet at 6 hours, they were well scoured and the coating of cement paste was removed from exposed pieces of aggregate. This evidently provided a much better condition for securing good bond than did the wire-brushing, probably because the brushing operation loosened the bedment of some of the aggregate, and it was evidently better than cleaning with the air-water jet at the later ages because it resulted in a rough-textured surface of clean aggregate.

TABLE 6—EFFECT OF METHOD OF CLEANING SURFACE OF LOWER LIFT UPON BOND STRENGTH

NOTES: Equivalent mix: 1.0 bbl./cu. yd.; special low-heat cement; mortar layer; 3-day interval between lifts.

Series	Slump, in.	Method of Compaction	Age at Test, Days	Modulus of Rupture, p.s.i.		
				Wire-brushing	Cleaning with Air-water Jet	
				at 12 Hr.	at 6 Hr.	at 20 Hr.
I	2½	Hand	28	102	365	...
			90	199	565	...
III	2½	Hand	28	283	...	340
	1½	Vibration	28	404	...	445
	½	Vibration	28	412	...	474

TABLE 7—EFFECT UPON BOND STRENGTH OF AGE OF CONCRETE AT TIME OF CLEANING SURFACE OF LOWER LIFT

NOTES: Equivalent mix: 1.0 bbl. per cu. yd.; in Series I—special low-heat cement; in Series II—blended low-heat cement; mortar layer; 3-day interval between lifts.

Series	Slump, in.	Method of Compaction	Age at Test, Days	Modulus of Rupture, p.s.i.			
				Cleaning with Air-water Jet			
				at 6 Hr.	at 8 Hr.	at 12 Hr.	at 20 Hr.
I	1½	Vibration	28	368	...	...	174
			90	576	...	...	217
II	3	Hand	28	462	415	...	...
	1½	Vibration	28	516	...	335	...

The relative effects of using a mortar layer as against no mortar on the joint, and using internal vibratory tamping vs. internal-plus-surface vibration are indicated in Table 8. Consulting the table it will be seen that on the average, the use of the mortar layer increased the joint strength by about 40 per cent at 28 days and by about 15 per cent at 3 months. The method of using the surface vibrator, in

TABLE 8—EFFECT UPON BOND STRENGTH OF (1) MORTAR LAYER AT BONDING SURFACE AND (2) INTERNAL VIBRATION VS. INTERNAL-PLUS-SURFACE VIBRATION

NOTES Special low-heat cement; 3-day interval between lifts.

Equivalent Mix, bbl./cu. yd.	Slump, in.	Vibratory Compaction Method	Modulus of Rupture, p.s.i.			
			Age at Test: 28 Days		Age at Test: 3 Months	
			No Mortar	Mortar Layer	No Mortar	Mortar Layer
1.0	1½	Int.	80	174	222	217
		Int. + Surf.	110	167	235	209
	½	Int.	177	189	236	269
		Int. + Surf.	137	143	160	298
0.8	1½	Int.	79	144	221	268
		Int. + Surf.	82	129	251	266
Avg. Bond Str., int. compaction only			112	169	226	248
Avg. Bond Str., int. + surf. compaction			110	146	215	258
Avg. Bond Str., both comp. methods			111	157	221	253

addition to the internal vibrator, resulted in a decrease in bond strength on the average of about 4 per cent.

That with the proper method of joint preparation the modulus of rupture along the joint plane compares quite favorably with the modulus of rupture of corresponding concrete placed by a continuous operation is indicated by the values of Table 9.

For example, in the tests of Series II, the strength of specimens with joints cleaned by the air-water jet at 6 hours was 96 per cent of the strength of corresponding plain specimens with no joint, while for the 8-hour cleaning this ratio of strength was 87 per cent. In Series III the flexural strength of specimens with wire-brushed joints varied from 55 to 78 per cent of the strength of corresponding plain specimens, depending upon the consistency of the concrete and the method of compaction, while the flexural strength of specimens with joints cleaned by the air-water jet at 20 hours varied from 66 to 85 per cent of the strength of corresponding specimens without joints. Also, it will be noted that the strength of joints cleaned by the air-water jet was substantially greater than the strength of corresponding joints cleaned by wire-brushing.

Effect of Method of Placement and Joint Preparation upon Placeability

The results of tests to determine the permeability of joints in general indicated a high degree of watertightness regardless of the method of placement and method of preparation of the joint surface. It is significant that among all the specimens tested actual leakage through

TABLE 9—COMPARATIVE FLEXURAL STRENGTHS OF CONCRETE PRISMS WITH AND WITHOUT JOINTS

NOTES: Mortar layer used on all joints.
In Series II: Maximum size of aggregate 6 in.
Cement: blended low-heat.
Prisms: 2½ by 2½ by 5 ft.
In Series III: Maximum size of aggregate 1½ in.
Cement: Special low-heat.
Prisms: 8 by 8 by 36 in.
Equivalent cement content, both series: 1.0 bbl. per cu. yd. Age at test: 28 days.

Series	Method of Cleaning	Method of Compaction	Slump, in.	Modulus of Rupture, p.s.i.		Ratio of Strength of Jointed Specimens to Strength of Plain Specimens
				Construction Joint	No Joint	
II	Air-water jet at 6 hr.	Hand	3	462	480	0.96
	Air-water jet at 8 hr.	Hand	3	415	480	0.87
III	Wire-brushed at 12 hr.	Hand	2½	283	518	0.55
		Vibrated	1½	404	521	0.78
		Vibrated	½	412	585	0.71
						Avg: 0.68
III	Air-water jet at 20 hr.	Hand	2½	340	518	0.66
		Vibrated	1½	445	521	0.85
		Vibrated	½	474	585	0.81
						Avg: 0.77

the joint occurred in two prisms only, and the joints of both of these were without mortar layer.

A study of the observed inflows over the 12-hour test periods indicates (a) that on the average the permeability was less for vibrated concrete than for hand-tamped concretes, (b) that greater variation in inflow as between individual specimens occurred for vibrated concretes of ½-in. slump than for vibrated concretes of 1½-in. slump, (c) that other things being equal, the joints with mortar layer were less permeable than those without mortar layer, (d) that the joints which were cleaned with the air-water jet were less permeable than those which were wire-brushed, (e) that joints cleaned with the air-water jet before the time of final set were less permeable than those cleaned with the air-water jet after the time of final set, and (f) that surface vibratory tamping following internal vibratory tamping tends to increase joint permeability.

Effect upon Bond Strength of Various Periods of Drying of Surface of Lower Lift

The effect upon bond strength of various periods of drying of the surface of the lower lift was investigated in Series III for periods up to 10 hours. For one group, the upper lift was placed at the end of a

TABLE 10—EFFECT UPON BOND STRENGTH OF VARIOUS PERIODS OF DRYING OF SURFACE OF LOWER LIFT—SERIES III

NOTE: Each value given in the table is the average of results of tests on concretes of 3 consistencies exposed to two drying temperatures.

Group	Average Modulus of Rupture at 28 Days, p.s.i.					
	Exposure to Dry Air for Period of					
	0 Hr.	½ Hr.	1 Hr.	2 Hr.	6 Hr.	10 Hr.
A. Upper lift placed at end of drying period; no mortar; no surface cleaning	541	448	404	429	357	335
B. Air-water clean-up at 20 hr.; mortar layer; interval between lifts 3 days	420	419	371	421	373	424
C. Surface wire-brushed at 12 hr.; mortar layer; interval between lifts 3 days	366	355	373	369	374	351

given drying period, while for the two remaining groups, the interval between lifts was 3 days.

In Table 10 are given data indicating the effect of the length of these drying periods upon the bond strength as measured by the modulus of rupture. Each value is an average for the given drying period within the group, of the results of tests on specimens of three consistencies and two drying temperatures.

Consulting the table it appears that except for Group A, where the concrete of the upper lift was placed directly upon the surface of the lower lift at the end of the drying period, for the drying periods here considered the drying had practically no effect upon the strength of the joint. For Group A, the effect of the drying of the surface is marked, and on the average the bond strength after 6 to 10 hours exposure is roughly 65 per cent of the strength of specimens made by casting continuously. This is a fact of considerable importance, since under job conditions delays in casting are likely to occur.

The results for Groups B and C indicate that the injurious effect of surface drying is overcome by the subsequent cleaning operations followed by moist curing of the surface until the time of placing the next lift.

Effect of Degree of Drying Temperature upon Bond Strength

The effect upon bond strength of the degree of the drying temperature of the draft of air to which the bonding surfaces were exposed is shown by the values of Table 11. A study of the table indicates that on the average, the degree of temperature had no marked effect upon the bond strength, although considering the various consistencies separately, it appears that for the wettest consistency the temperature of 130° F. resulted in higher strengths, while for the driest consistency, the higher strengths were obtained from specimens subjected to the 100° F. temperature. This appears reasonable in that the concretes

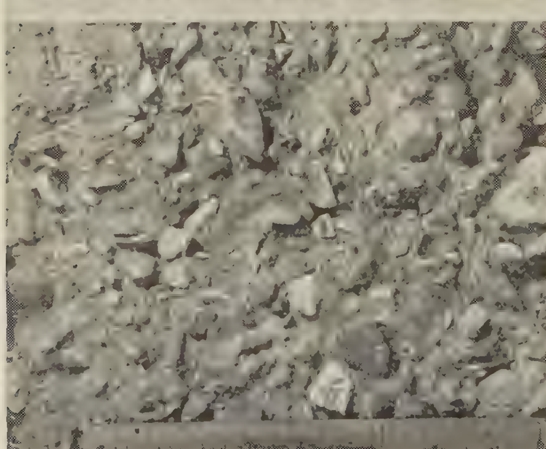


FIG. 1—SURFACE CLEANED WITH AN AIR-WATER JET BETWEEN INITIAL AND FINAL SET—ABOUT SIX HOURS

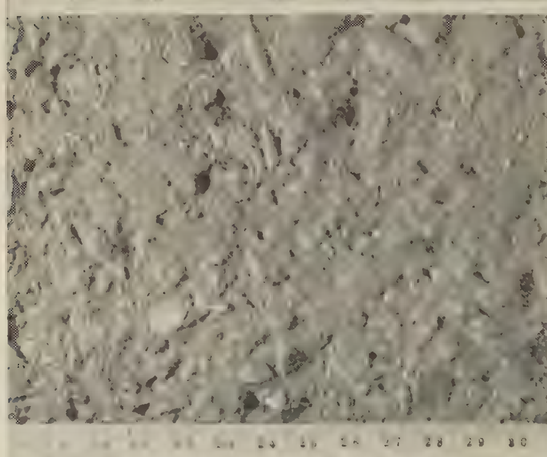


FIG. 2 SURFACE WIRE-BRUSHED AFTER FINAL SET

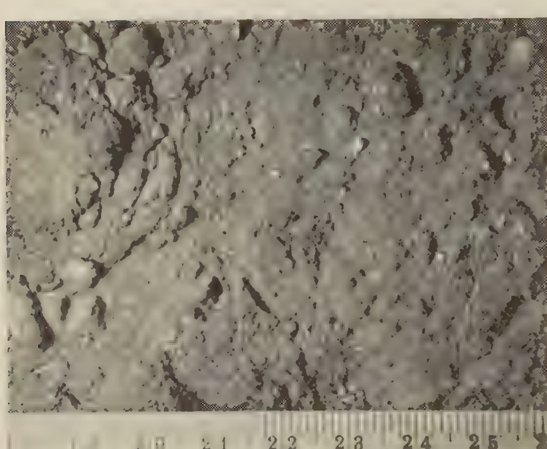


FIG. 3—SURFACE CLEANED WITH AN AIR-WATER JET AFTER FINAL SET—20 HOURS

of 2½-in. slump had some excess surface moisture after compacting, while the concretes of ½-in. slump carried very little excess moisture.

TABLE 11—EFFECT OF DEGREE OF DRYING TEMPERATURE UPON BOND STRENGTH—SERIES III

Group	Slump, In.	Modulus of Rupture at 28 Days, p.s.i.	
		Drying Temperature 100° F.	Drying Temperature 130° F.
A. Upper lift placed at end of drying period; no mortar; no surface cleaning.	2½	359	368
	1½	379	379
	½	454	433
	Avg:	**397**	**393**
B. Air-water clean-up at 20 hr.; mortar layer; interval between lifts 3 days.	2½	360	391
	1½	409	410
	½	436	404
	Avg:	**402**	**402**
C. Surface wire-brushed at 12 hr.; mortar layer; interval between lifts 3 days.	2½	358	366
	1½	362	339
	½	387	374
	Avg:	**369**	**360**
Average for all 2½-in. slumps:		**357**	**375**
Average for all 1½-in. slumps:		**383**	**376**
Average for all ½-in. slumps:		**426**	**404**
Average for all conditions:		**389**	**385**

Effect of Time Interval between Lifts upon Bond Strength and Permeability

The effect of time interval between lifts upon bond strength and permeability along the plane of the joint is indicated by the results shown in Table 12, where are given values of the modulus of rupture and the average inflow over a 12-hour period for specimens having intervals between lifts of 3 and 12 days.

Consulting the table it will be seen that for the given methods of placement and for the given cements, with few exceptions the permeability was lower and the joint strength was higher for the 12-day interval between lifts than for the 3-day interval between lifts. It should be noted that the joint surfaces were kept continuously moist.

Effect of Type of Cement and Richness of Mix upon Bond Strength and Permeability

Results of tests to determine the effect of type of cement and richness of mix, which have been shown in Tables 5 and 12, indicate that

TABLE 12—EFFECT OF TIME INTERVAL BETWEEN LIFTS UPON BOND STRENGTH AND PERMEABILITY—SERIES I

NOTES: Equivalent mix 1.0 bbl. per cu. yd.
Mortar layer.
Concrete of 2½-in. slump; hand-tamped; joint cleaned by wire-brushing.
Concrete of ½-in. slump; internally vibrated; joint cleaned with air-water jet.

Cement	Slump, in.	Age at Test, Days	Modulus of Rupture, p.s.i.		Permeability, Avg. C.C. per Hr. for 12-Hr. Period at 100 p.s.i.	
			3-Day Interval	12-Day Interval	3-Day Interval	12-Day Interval
Special low-heat	2½	28	102	171	65	12
		90	199	165	44	9
	½	28	189	155	32	19
		90	269	388	20	35
Standard	2½	28	166	212	20	12
		90	241	422	12	14
	½	28	257	261	20	16
		90	298	500	17	7

at the age of 28 days the more rapid hardening standard portland cement or the richer mix produced greater bond strength and lower permeability than did the slower hardening low-heat cement or the leaner mix. A choice with respect to these factors would be governed by other considerations than their joint-bonding qualities alone, although it is significant that the joint permeability is lower and the bond strength is higher for concretes of 1½ or ½-in. slump, containing 0.8 bbl. of cement per cu. yd. compacted by vibratory tamping and having the joint surfaces cleaned with high pressure air-and-water, than for concretes of 2½-in. slump containing 1 bbl. of cement per cu. yd., placed by hand tamping and having the joints prepared by wire-brushing. Considerations such as this would appear to point the way toward important economies.

CONCLUSIONS

The following general conclusions are based upon the results of the tests, made under the conditions stated.

1. Compaction by internal vibration without cleaning other than by use of an air-water jet, results in a distinctly higher joint strength and greater degree of joint impermeability than does compaction by hand tamping followed by wire-brushing.

2. The optimum time for scouring the surface of the lower lift with the air-water jet appears to be that at which the cement paste can be washed from the surfaces of projecting pieces of aggregate. Regard-

less of the consistency of mix and method of compaction, this procedure is superior to wire-brushing. When the surface of the lower lift is cleaned with the air-water jet just before final set (6 to 8 hours after placing the concrete), the ratio of flexural strength of specimens with joints to flexural strength of plain specimens without joints is roughly 90 per cent. For specimens employing the air-water cleaning after final set (age 20 hours) this ratio is on the average about 77 per cent, while for specimens employing the wire-brush method shortly after final set (age 12 hours), the average ratio is 68 per cent.

3. The use of a surface type of vibrator, in addition to the internal type of vibrator, to embed large pieces of aggregate which would otherwise protrude from the mass, appears to have no beneficial effect. On the contrary, its use seems somewhat to decrease the joint strength and impermeability.

4. The harmful effect of exposure of the bonding surface to a drying atmosphere of 100 or 130° F. appears to manifest itself principally when the upper lift of fresh concrete is deposited directly upon the dried surface, a condition such as would occur due to delays in placing on construction work. If the dried surface is cleaned and then kept moist until the next concrete is cast, no harmful results are evident.

5. Within the limits of the tests (100 to 130° F. with drying periods up to 10 hours) variations in the air temperature apparently have little influence upon the joint strength except for the driest mix, for which mix the bond strengths are adversely affected by the higher temperature.

6. The procedure of treating the joint with a layer of mortar just prior to placing the concrete of the upper lift is distinctly to increase the joint strength. Also, while the average effect upon permeability appears to be negligible, among the individual joints tested those exhibiting imperfections as indicated by leakage upon the application of water pressure were without the mortar layer.

7. Where the surface of the lower lift is kept continuously moist until the upper lift is placed, the bond strength and impermeability are on the average somewhat improved by extending the interval between lifts from 3 days to 12 days, but the difference is not marked.

8. The average bond strength and impermeability, as well as the compressive strength, are considerably greater for a concrete of 1½-in. slump containing 0.8 bbl. of cement per cu. yd., placed by vibration

without wire-brushing than for a corresponding concrete of 2½-in. slump, containing 1.0 bbl. of cement per cu. yd., placed by hand tamping with the surface of the lower lift wire-brushed. This applies to joints where the old surface is flushed with mortar prior to placing the new lift.

9. Within the limits of these tests, it appears that concrete of 1½-in. slump, placed by internal vibration, possesses properties which, all things considered, are superior to those of corresponding concrete of 2½-in. slump placed by hand tamping or ½-in. slump placed by internal vibration. The advantages of the 1½-in. slump with vibratory compaction are: (a) ease of placement, as indicated by time studies, (b) greater joint strength and impermeability than for concrete of wetter consistency (2½-in. slump) compacted by hand, (c) the lowest degree of absorption of the concretes of the three consistencies considered, and (d) in general, the least variation in joint permeability between individual joints, and hence the greatest degree of certainty with respect to the obtaining of thorough compaction.

For such discussion of this paper as may develop, readers are referred to the JOURNAL *for November-December 1934 (Proceedings Vol. 31). Discussions should be available to the Secretary by October 1, 1934.*

Does Cement Protect a Poor Quality Aggregate?—Yes and No*

BY E. VIENS†
MEMBER AMERICAN CONCRETE INSTITUTE

The title of this paper may seem to contradict itself. However, I think I can show that some aggregate will cause a 2000 lb.-28 day concrete to deteriorate much sooner than would a concrete of like design with a much better aggregate. Yet, some aggregate not durable in a 2000 lb. concrete, will give entire satisfaction in a 3500 to 4000 lb. concrete.

This paper is not concerned with concrete which is not exposed to the elements, for many an aggregate not satisfactory in an exposed structure might be entirely satisfactory in buildings where the concrete is not subjected to moisture and extreme variations of temperature.

Four different aggregates are dealt with in this paper.

CASE 1

Fig. 1 to 5 show a 3000-lb. concrete in which 6¾ cu. ft. of cement were used per cubic yard. The aggregate was the debris of various formations of stone, from granite to comparatively soft sandstone and shale. The percentage of shale and soft sandstone was approximately 12 per cent. In its large particles, the sandstone was responsible for the larger cracks, and the small particles of the sandstone and shale were responsible for the crazing as shown in these illustrations.

In Fig. 1 two jack-knives are shown stuck in pieces of soft sandstone (about 1¼ in. diameter) which exerted sufficient pressure to spall the concrete and push off the stucco.

The ramp wall in Fig. 2 shows the effect of a soft sandstone which caused a small slab of concrete to spall.

Fig. 3 shows the slab itself.

In Fig. 4 the middle step shows, apparently, the effect of a large unsound sandstone, as it will be seen the crack is of considerable proportions.

*Presented at the 30th Annual Convention. American Concrete Institute, Toronto, Feb. 20-22, 1934.
†Director, Testing Laboratories, Public Works Department, Ottawa.

FIG. 1, 2, 3—(TOP, BOTTOM, RIGHT)—AN AGGREGATE FAILURE DISCUSSED IN "CASE 1"

Fig. 5 shows similar effects, the radiating cracks showing approximately the location of bad stone.

The concrete illustrated in these five pictures was made in late October and early November, 1927. It was well protected from any frost action and was allowed to mature under damp conditions for about three weeks but did not have time to get rid of water in excess of that required for hydration, before the severe cold weather set in. The soft, porous sandstone being wet, no doubt did the damage very early after the cold weather set in, for the following spring, most of the cracks seen in the illustrations were visible. Had this same concrete been made in early May instead of late fall, properly matured and thoroughly dried previous to the heavy frost, it might have stood very much better, since the concrete was sufficiently rich in cement to make it more or less watertight, but no doubt the soft sandstone pebbles nearest to the surface, would, in time, have got in their work.

FIG. 4-5—DISCUSSED BY THE AUTHOR UNDER "CASE 1"

CASE 2

The concrete in a group of scattered stretches of sidewalks failed as shown in Fig. 6, 7 and 8.

These sidewalks were built in October and early November, 1930, and the failures were first noticed the following May. The sidewalks were laid in two courses, the bottom 4 in. course of a 1:3:6 concrete using an aggregate up to ¾ in. diameter, and a top wearing course, 1½ in. thick of a 1:2:4 concrete using an aggregate up to ⅜ in. diameter. These actual proportions were not followed but an equivalent amount of cement was used in each case and the ratio of coarse to fine aggregate was varied to give a concrete that would place and finish well. Because of a desire for a good wearing surface, the top course was slightly undersanded. The aggregates used were washed sand and gravel of good grading. The gaging water used was from

FIG. 6, 7, 8—SEE "CASE 2"

the city mains. The consistency was 4- to 6-in. slump and uniform. The approximate water-cement ratio of the 1:3:6 concrete was 1.45 and of the 1:2:4 concrete 1.2 corresponding to a compressive strength at 28 days of 1300 and 1900 p.s.i. respectively.

A distinct increase in volume was noticed in several pebbles causing cracks similar to that shown in Fig. 6 at the near end of the lath. Noticeable popping of the aggregate was found, as in Fig. 6 at the end of the pencil on the left-hand side, and also at several places in Fig. 8. In every instance the stones at the bottom of these holes were greenish in color and all were of a similar texture. Actual disintegration had already started along the edges in a few places, as shown in Fig. 7 and 8.

All of the sidewalks investigated were laid by one contractor and the concrete was purchased by him ready mixed. An investigation showed that all of the defective concrete came from one of the two ready mix plants delivering concrete to this contractor and also showed that the trouble coincided with the use of a particular aggregate at this plant. During the interval over which this trouble occurred the aggregate used at this plant was not from the usual supply and it was found that the worst failures occurred immediately after shipments of this latter aggregate were received and sporadic trouble was found in all the sidewalks laid after beginning the use of this aggregate. The defective aggregate was mainly of a ¾-in. size, and, therefore, was used in the bottom course. However, there was one shipment of ⅜ in. gravel from the same source and the sidewalk in which this was used showed extensive popping in the top course, as in Fig. 8. Most of these sidewalks have since been replaced.

Soundness tests were made on selected pieces of the greenish colored stone found in this gravel, using the sodium sulphate test of the U. S. Bureau of Public Roads. Of 67 pebbles tested, 58 failed in the first cycle, and by the fourth cycle, 65 had failed. It was also found that some of these greenish pebbles would fail in merely being wetted and dried once, using plain water.

A petrographic examination of the defective particles, both from the sidewalks that failed and from the soundness test, showed them to be similar. mostly of dolomitic limestone sometimes bordering on a shale and described as weak structurally showing loose bonding, unevenness of grain, and inclined to be schistose in structure.

Soundness tests made on similar aggregates from the same pit some months prior to its use in these sidewalks had not shown the presence of any dangerous amount of unsound particles and apparently

the character of the pit had changed in the meantime. Rock of the kind that gave trouble is known to exist in many of the pits in operation over a wide area but never previously in quantities that had caused trouble. It was found that other sidewalks laid about the same time by other contractors using the same gravel developed similar failures.

Some of this aggregate was used in a 3000-lb. concrete made in the same ready-mix plant and after three years the concrete is in excellent condition with no sign of any trouble developing due to the aggregate or other causes.

CASE 3

This case is in connection with a concrete road, 9 or 10 miles long, built in July and August of 1931. In the summer of 1932 this section of road was inspected by two well known concrete authorities and nothing out of the ordinary was noticed. It was re-examined by one of the two authorities in 1933 and it was discovered that many poppings similar to that at the point of the hammer of Fig. 9 were taking place. At the bottom of each popping hole was a greyish-white stone, badly shattered, as shown in Fig. 10. Elsewhere on the road surface could be found dozens of incipient poppings as shown in Fig. 11. Many cracks have also developed in the concrete as can be noticed in these illustrations.

In one section of 750 sq. yds. of surface, 49 poppings of all sizes from one inch up were found, and in addition, 89 incipient poppings, which, on opening up, disclosed the characteristic greyish-white stone. In the worst section the concrete was found to be badly cracked in all directions but due to the high quality of the concrete the cracks were fine and have not yet ravelled at the edges.

The proportions for the concrete of this road were 1:2:3; maximum size of coarse aggregate about 2½ in.; 6.7 sacks of cement were used per cubic yard, and the concrete had an approximate water-cement ratio of 0.9. Its average compressive strength at 28 days was 4200 p.s.i. All aggregate was washed and screened.

The aggregate had been given the soundness test mentioned above before concreting started and it passed the usual five cycles satisfactorily. This aggregate was almost all composed of the debris of limestone and dolomitic limestone containing some chert. The significant feature in this case is the failure of an aggregate in a 4000 lb. concrete and the serious cracking that is occurring, and there is no question as to the care that was exercised in the manufacture and placing of the concrete in question.

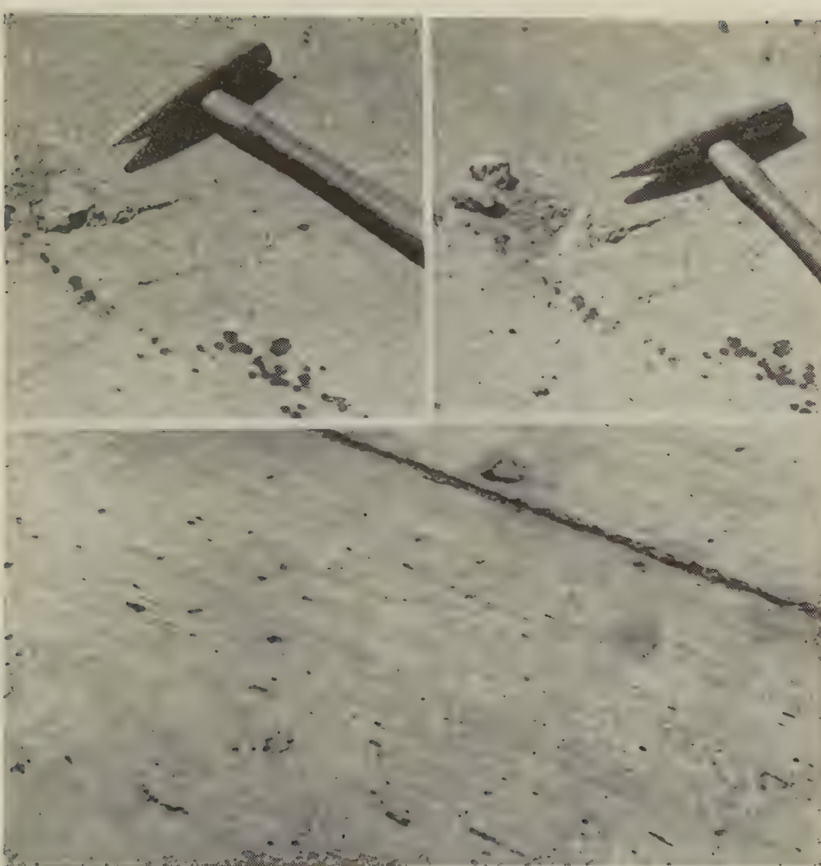


FIG. 9, 10, 11—(TOP, RIGHT, BOTTOM)—SEE "CASE 3" OF CONCRETE AGGREGATE FAILURE

CASE 4

This case deals with the concrete of a harbor on Lake Huron, Canada, in which the first concrete structure was part of what is called the Northwest breakwater, built in 1908. A few years later, this breakwater was extended to its present length. During the years 1911 to 1917 another breakwater was built—the Southwest breakwater. Between the years 1923 and 1926 the old wooden breakwater at the mouth of the Maitland river was replaced with concrete and the north and south piers which serve as an entrance to the harbor, and which were also constructed of wood, were replaced by concrete between the years of 1923 to 1932.

The aggregate used for all of this concrete was taken from the shore of the lake near the work, the aggregate consisting almost entirely of the debris of various formations of limestone, running from fairly pure limestone to dolomitic limestone, some containing chert and a form of iron, and a fairly high percentage of the stone being porous and structurally weak. The older concrete of this harbor, that is, the Northwest and Southwest breakwaters, contained approximately 5 to 5¼ cu. ft. of cement per cu. yd., and yielded a concrete in the neighborhood of 1800 to 2000 p.s.i., while the newer concrete from 1923 to 1932 contained 7 to 7¼ cu. ft. of cement per cu. yd. yielding a concrete of about 3500 p.s.i. The older concrete is disintegrating very seriously. Some of it had to be repaired nine years ago to prevent the possible loss of the whole of the older part of that concrete. The failure of the concrete indicates that the aggregate is largely responsible for the failure, for we know of other structures built about the same time with approximately the same amount of cement but with much better aggregate that are standing fairly well today, although very few people, I believe, in this present day, would advocate the use of 2000 lb. concrete for exposed structures. Before replacing the old wooden breakwater and the two wooden piers, the question of importing an aggregate from some other source was considered, but after considerable study of the matter it was found that it would be more economical and satisfactory to use much larger quantities of cement than had been used previously and to continue with the same aggregate.

Fig. 12 and 13 show views of the Northwest breakwater built in 1908. It will be observed in Fig. 12 how a large number of the pebbles in the aggregate split with the action of moisture and frost.

Fig. 14 shows a close view of the Southwest breakwater, and again it will be observed that many of the pebbles are split up.

Fig. 15 shows part of the river breakwater at the mouth of the Maitland river and it will be observed that the concrete does not exhibit the least sign of failure after from eleven to eight years of exposure.

During the construction of these latter structures, that is, the river breakwater and the north and south piers, the aggregate was continually tested and the mix to be used on the job was designed in a laboratory. The laboratory-made specimens yielded an average strength of 3500 p.s.i. Field specimens were taken at frequent intervals during the construction, the average strength of which was over 3000 p.s.i.

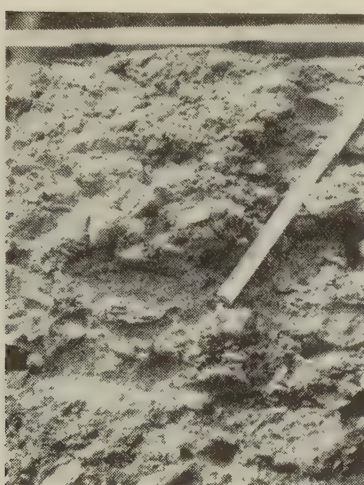


Fig. 12—"Case 4"

In 1928 a large number of cores of 5⅝ in. diameter, and approximately one foot long, were taken in various parts of the new structures, yielding an average strength of 4600 p.s.i. Cores were also taken from the older structures below the surface disintegration, and the average strength of these cores was 3000 p.s.i.

If concrete in moist condition keeps on curing from 8 to 10 years, as some authorities point out, these older structures certainly had time to acquire their maximum hydration and strength. Some of the newer structures from which cores were extracted were only two years old at the time of the extraction of the cores, and, therefore, had not reached their maximum hydration and strength.

From the appearance of the concrete in the newer structures at this time of writing and results obtained, as described above, the author feels confident that this aggregate can be protected with cement to give entire satisfaction.

CONCLUSIONS

It will be observed from the four cases mentioned in this paper that two of the aggregates, that is, cases 1 and 3, in the opinion of the writer, could not be sufficiently protected economically by cement to give satisfaction in exposed structures. The aggregates of cases 2 and 4, no doubt, can be sufficiently protected to give satisfaction, and it will be observed that in case 1, the failure of the concrete is due to

FIG. 13, 14—"CASE 4"

sandstone and shale, while in case 3, the failure is due to certain types of dolomitic limestone.

An aggregate should not be rejected because it contains sandstone or limestone, for there are certain acceptable sandstones and we know very well that much of the limestone is also acceptable. Each deposit should be studied by itself and carefully watched from time to time as it is possible for some deposits to vary in such a way as to be unreliable as in case No. 2.

A great urgent need in the technique of testing aggregates is the development of a reliable accelerated test that will determine the

Fig. 15—"Case 4"

quality of an aggregate for exposed concrete. For concrete which is to be permanently protected from the elements, the question of the aggregate to be used is an entirely different one. An aggregate might be rejected for exposed concrete while it might be entirely satisfactory for other purposes.

The question as to whether an aggregate should be used or not used is also one of local economy, as to whether more cement should be used, if the aggregate can be protected by cement or another aggregate brought in from a distance.

In the case of an aggregate which can be protected by using larger quantities of cement, it might be advisable, to get *further protection* to limit the size of the aggregate to a 1½-in. maximum so that less force would be exerted by the least sound pebbles.

I believe that the four cases discussed above justify the title of this paper.

For such discussion of this paper as may develop, readers are referred to the Journal *for November-December 1934 (Proceedings Vol. 31). Discussions should be available to the Secretary by October 1, 1934.*

Tests of Reinforced Concrete T-Beams*

BY T. D. MYLREA†

MEMBER AMERICAN CONCRETE INSTITUTE

In the early days, we are told, bridges were often built before the plans were finished, and frequently the spans had to be strengthened or entirely replaced. In some respects the same thing has occurred in the field of reinforced concrete. We were suddenly confronted with a wonderful new building material, and structures had to be built. Governmental bureaus and Universities did their best to supply the urgent demand for statistical information, and a theoretical approach to the subject of design was evolved. It is now evident that some of our formulating proceeded too rapidly, and we are now very busily engaged in the difficult task of revising, to conform with more recent ideas, some of the conceptions which we first thought were inviolate.

Among the first to protest against too rapid crystallization of ideas were Edward Godfrey and William Fry Scott, and even at this date Mr. Godfrey's paper of 1904 and Mr. Scott's paper of 1910 make instructive reading. It may be possible that both men would take back some of the statements they made at that time, but nevertheless there is much of value in their papers.

Mr. Scott was the first to question in print the sanctity of the bond relationship, and in 1920 he made some demonstrations of a system he had devised, in which the troublesome questions of bond and shear were eliminated. The writer was called in as a disinterested witness. Mr. Scott's beams were much stronger than had been anticipated, but he steadfastly refused to explain how he designed them.

Since that time the writer has been especially intrigued by the subjects of bond, shear, and anchorage. Many of the results of this study have appeared in print,[1] but because of the bearing of the conclusions on the building code we are now formulating, and in

*Presented at the 30th Annual Convention, American Concrete Institute, Toronto, Feb. 20-22, 1934.
†Professor of Building Construction, Carnegie Institute of Technology, Pittsburgh.
[1]Jour. West. Soc. Eng.—Jan. 1926.
Trans. Am. Soc. C. E.—1930, p. 734.
Proc. Amer. Concrete Inst., Vol 24, 1928, p. 240.

order to prove or disprove the statements previously made, the series of tests described in this paper was planned. In describing Mr. Scott's system the writer is violating no confidence, because no confidences were ever given. At this late date acknowledgment of the originality of some of the features in his scheme of reinforcement should be sufficient.

In the Scott system of reinforcement small bars are used. These are bent, two pairs at a time, at points where they are no longer needed to resist moment. The radius of bend is such that at middepth of the beam the bars become tangent to lines making an angle of 45 deg. with the axis of the beam. The bars are also bent transversely across the web and back again into the flanges, where each pair is joined by a dished anchor plate. About 20 per cent of the bars do not reach the flanges but are carried through to the end of the stem, where they, too, are anchored in pairs. The bars nearest the neutral axis are generally not the ones to be bent first, for Mr. Scott believed that the interweaving of bars contributed to the strength of the beam. A better idea of the placing of the steel may be obtained by reference to Fig. 1 and Fig. 9. Prof. Richart has since presented a neat proof that when relatively small bars are used the number will be sufficient for moment as well as for complete web reinforcement. (See Univ. of Ill. Bull. No. 166, p. 20.)

The beams of the series tested consisted of five T-beams of ten foot span. The width of the flange was 21 in.; the overall depth was 20 in.; the thickness of the flange and of the stem were each 4½ in. All beams were reinforced with the same amount of steel, 0.62 per cent and in all beams the effective depth to the center of gravity of the steel was kept the same. Beam No. 3 was reinforced with five ¾-in. round bars, two of which were carried through straight, the others being bent to form web reinforcement, as shown in Fig. 7. All the other beams were reinforced with 20—⅜-in. round bars. In beam No. 2 nine bars were carried through straight and the others were bent in orthodox fashion across the web for shear reinforcement. (See Fig. 5) In beam No. 1 Mr. Scott's scheme of bending was followed, the ends of all bars being hooked, as shown in Fig. 2. In the first three beams cold drawn steel bars containing about three and one half per cent nickel and having an elastic limit of about 70,000 p.s.i. were used. The results of these tests were so encouraging that an effort was made to get even stronger steel, and through the courtesy of the Concrete Reinforcing Steel Institute, ⅜-in. bars having an elastic limit of 96,000 p.s.i. were obtained for use in beams No. 4 and No. 5.

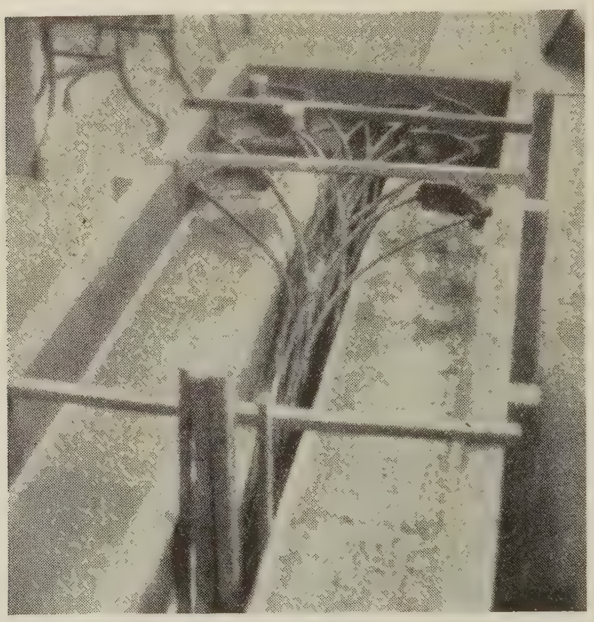


FIG. 1—SCOTT SYSTEM OF BENDING AND ANCHORING BARS

Beam No. 4 was a typical Scott beam, with anchors, and the details of the reinforcement are given in Fig. 9. The placing of the reinforcement is not so difficult as may be imagined, but it was felt that nevertheless a distinct gain would be possible if the interlacing of bars could be omitted in stems of limited thickness. It was therefore decided to test a beam in all respects like No. 4, except that interlacing of the bars was eliminated. This beam, No. 5, is shown in Fig. 11.

The concrete used in the first three beams had a cylinder strength averaging 5,500 p.s.i. In one sense this is unfortunate, for all failures were due to details of the reinforcement or to deflection caused by the great yielding of the steel after the elastic limit had been passed. The writer feels that the beams would have been equally strong if the strength of the concrete had been only half as great. We still have much to learn about the influence of concrete strength on beam strength.

All the beams were tested with two concentrated loads, 2 ft. 6 in. apart, placed over the stem of the beam only, and the results are shown graphically in the accompanying illustrations. Each load was applied and released 5 times, for it was felt that the racking effect thus produced was much more severe and more likely to bring out

inherent defects in the design than if a single load had been applied steadily to destruction. The crack pattern produced by the loading is shown on each beam diagram, the small numbers beside the cracks referring to the load number at which each crack became visible to the naked eye. The larger figures in circles are the gage line numbers. The deformation and deflections produced in each beam are shown immediately below the diagram of the beam itself. For convenience, the loads are given in tons of 2000 lbs., and in small figures near the left margin of the deformation diagrams are shown the load numbers.

The gage lines were so arranged that a number of problems could be investigated simultaneously; for example, on beam No. 1, G.L. 60, 58, and 32 were on the same bar at points near the support, under the load, and approximately midway between these two points. In this way the variation in stress along a single bar would become apparent. G.L. 48, 54, and 56 are all on one bar, but not on the same bar as the previous group. G.L. 30, 34, and 32 are on the upper, lower, and one intermediate bar on the same side of the beam, directly below one of the loads, and the readings on these bars were checked on the opposite side of the beam at G.L. 45 and 47. Under the other load, G.L. 46, 50, 48, 31 and 33 give similar information.

A study of the graphs reveals some very interesting information. If for convenience the value of E be assumed to be 30,000,000 p.s.i., each horizontal division corresponds to a unit stress of 30,000 p.s.i. The steel in beams No. 1, 2, and 3 being a drawn product, its elastic limit was not well marked, but its approximate value of 70,000 p.s.i. corresponds to 2⅓ horizontal divisions. On each deformation line this point is indicated by a short vertical line, and the load at which it occurred can be found by tracing horizontally to the left hand margin. Similarly the short vertical line in the graphs for beams No. 4 and 5 indicate an elastic limit of 96,000 p.s.i.

BEAM TESTS

Beam No. 1. It will be noted that with the reinforcement placed as it was in this beam, it was possible to apply a load which produced stresses far in excess of 70,000 p.s.i. in the steel. It will be noted too, that at G.L.60 the stress reached a value higher than the elastic limit, although at G.L. 56, this stress was not attained. Again comparing G.L. 30, 32, and 34, it is seen that the stress in the lower bar, while somewhat greater than that in the others, did not correspond to what might have been expected in a straight line distribution, which indicates that the usual practice of assuming that the center of gravity of tensions corresponds to the center of gravity of the reinforcement is not in error serious enough to cause concern. G.L. 60 gives an interesting example of the effect of a crack in the concrete. Between loads No. 7 and 8 stress was first discernible at this gage line, when an increase of

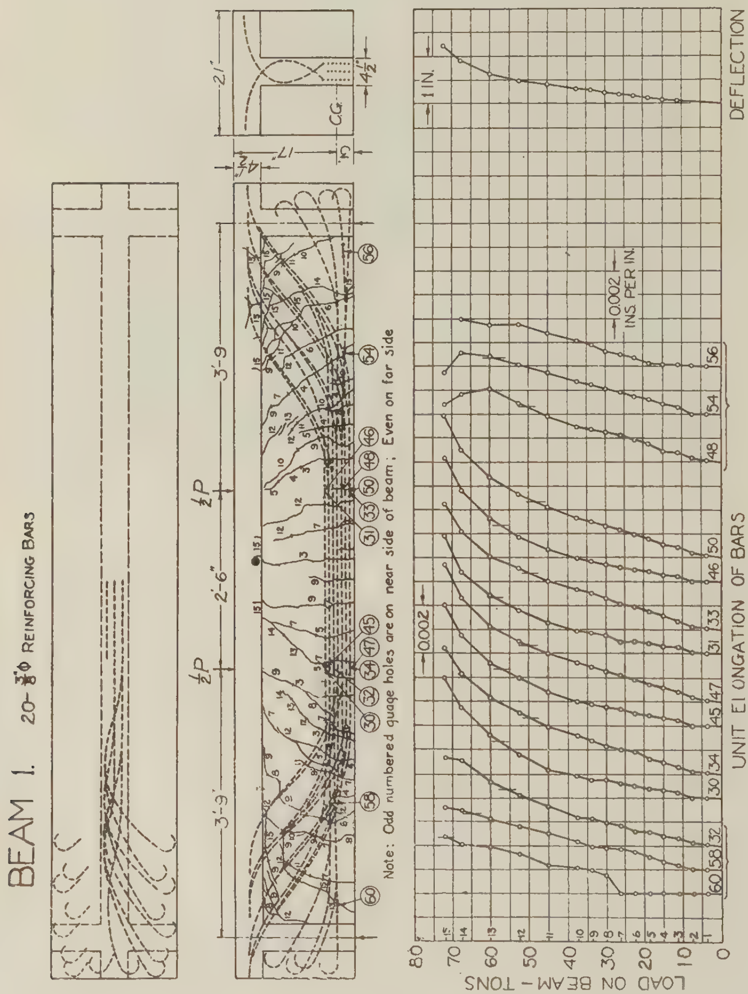

BEAM 1.
20-⅞"φ REINFORCING BARS
Note: Odd numbered guage holes are on near side of beam; Even on far side
LOAD ON BEAM - TONS
UNIT ELONGATION OF BARS
DEFLECTION
0.002 INS. PER IN.

FIG. 2

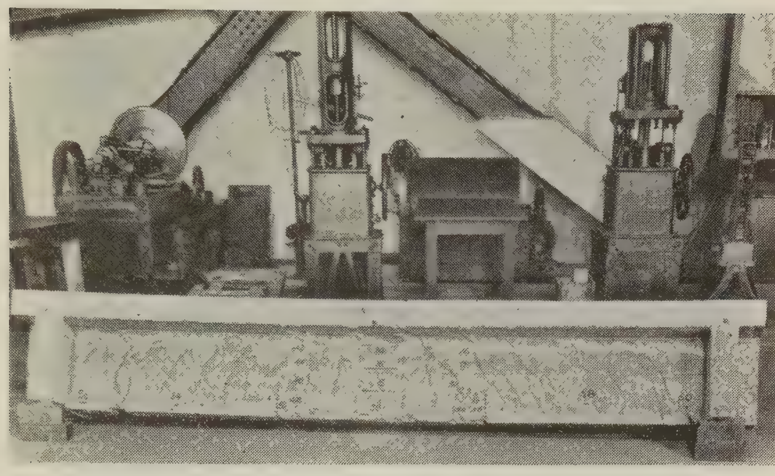
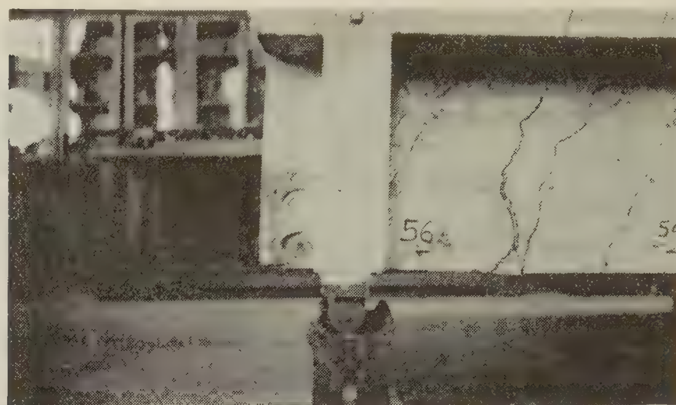


FIG. 3—BEAM NO. 1 AT FAILURE

approximately 20,000 p.s.i. suddenly became apparent. The figure above shows that at load No. 7 a crack appeared within 6 in. of this gage line. The crack pattern is very interesting, the general direction corresponding very closely to the stress trajectories which might be expected in a homogeneous beam of these dimensions. This indicates that the disposition of the reinforcement is such as to cause two materials, very unlike in physical properties, to act together to produce an almost homogeneous specimen.

The unit shear developed at failure in beam No. 1, as computed by the formula $v = \frac{V}{bjd}$, was 1050 p.s.i. Failure, however, was not due to the high shear stress, but to the crushing of the concrete under the hooks at the end of the beam. (The four hooks shown at the right hand end of the beam in Fig. 2.) This occurred at load No. 15 and was accompanied by the formation of a wide crack close to G.L. 56. The failure may be seen in Fig. 3 and 4.

Beam No. 2. Beam No. 2 was reinforced with the same number of bars of the same size as used in No. 1. The bending, however, was intended to correspond more nearly to conventional practice. Here again the gage lines are grouped to bring out interesting phenomena. The grouping is indicated by the brackets underneath the deformation diagram. From G.L. 5, 15, 25, 43, and 1, it is possible to trace the stress in one bar from the point of maximum moment to a point near the end of the bar. The stress at G.L. 5 is practically the same as at G.L. 15, but the stress at G.L. 43 is consistently less. Stresses in the upper portion of the bent bars, i.e., at G.L. 1 and 2, were not apparent until load No. 5 or 6, but before failure occurred a stress of 37,000 p.s.i. was apparent at G.L. 1, while the stress at G.L. 2 exceeded the yield point of the material. The influence of the formation of a crack near G.L. 45 at load No. 5 is very apparent, and the same effect is noticeable at many other points. The stress in G.L. 27 does not differ materially from that at G.L. 29. This is what might be expected from the close correspondence of the stresses at G.L. 27 and 45, except at high loads, which indicates that until these high loads were attained the



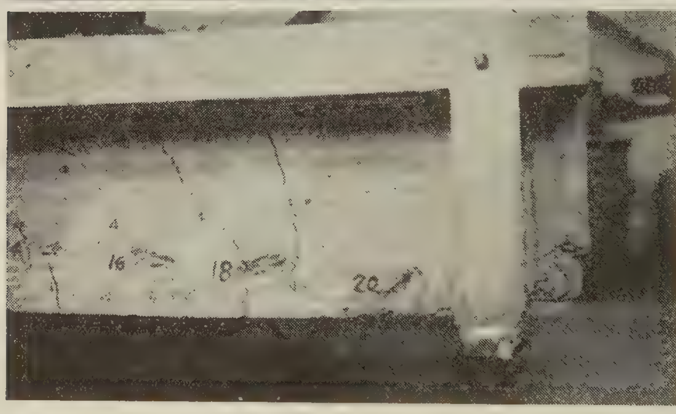


FIG. 4—DETAIL OF CRUSHING IN BEAM NO. 1
FIG. 6—BEAM NO. 2. CONCRETE CRUSHED UNDER HOOKS

bend between G.L. 27 and G.L. 45 did not act as an anchor to any appreciable extent. Failure occurred at a load of 98,000 lb., and was caused by crushing of the concrete beneath the hooks on the straight bars at the right end of the beam, Fig. 5, and is shown in detail in Fig. 6. The maximum unit shearing stress was 712 p.s.i.

The crack pattern is this beam bears very much less resemblance to the stress trajectories in a homogeneous beam than does the crack pattern in beam No. 1, thus indicating a much less effective reinforcement.

Beam No. 3. Beam No. 3 was designed in accordance with conventional practice, using 5 ¾-in. round bars, three stirrups being inserted near each load fully to reinforce the web, as shown in Fig. 7. Stress measurements were taken on these stirrups, but since no measurable stress was observed in them at any load, the data are not recorded on the deformation diagram. Here again, stresses were traced along a single bar from the point of maximum moment to a point as near as possible to the end, and in addition, the slip of the bars was measured at points A and B. In this beam the elastic limit was not reached at any point in the steel, failure due to crush-

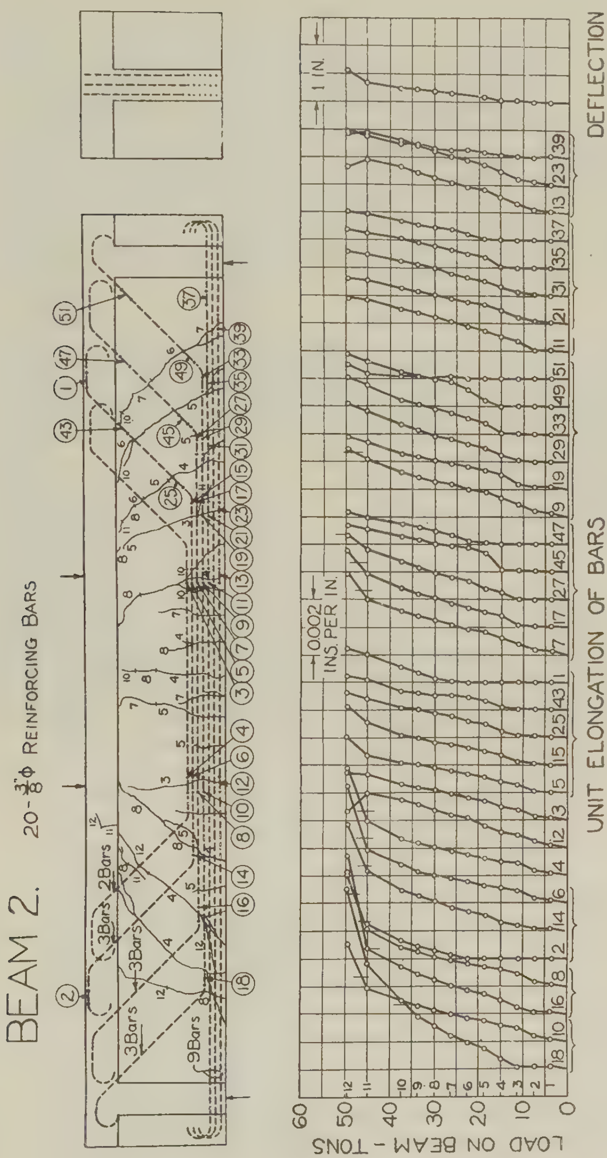
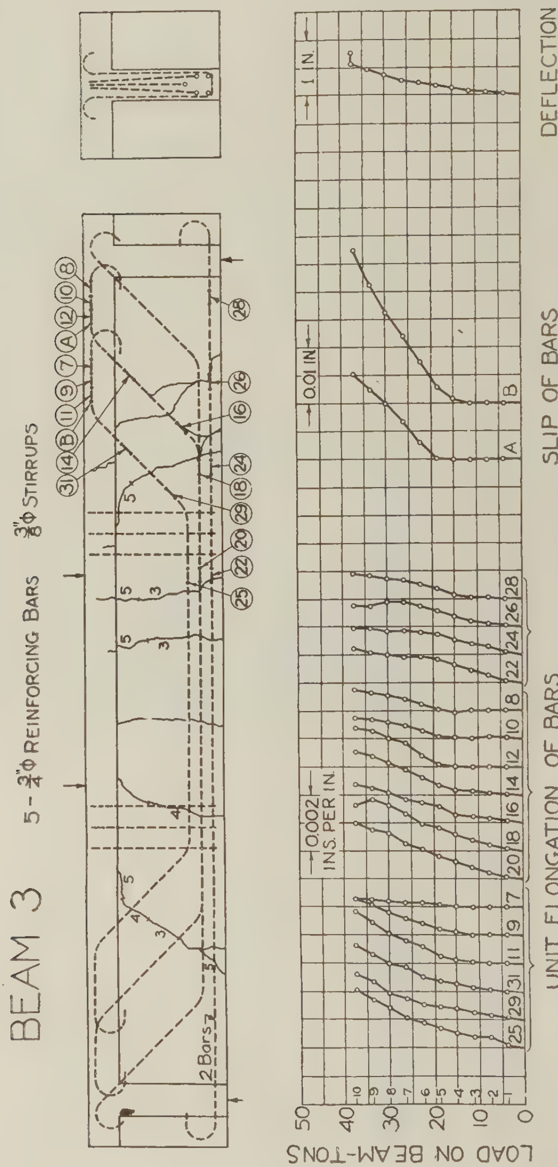

BEAM 2.
20-3/8"Φ REINFORCING BARS
0.0002 INS. PER IN.
1 IN.
LOAD ON BEAM - TONS
UNIT ELONGATION OF BARS
DEFLECTION

FIG. 5



BEAM 3
5 - 3/4"Φ REINFORCING BARS
3/8"Φ STIRRUPS
2 Bars
LOAD ON BEAM-TONS
0.002 INS. PER IN.
0.01 IN.
1 IN.
UNIT ELONGATION OF BARS
SLIP OF BARS
DEFLECTION

FIG. 7

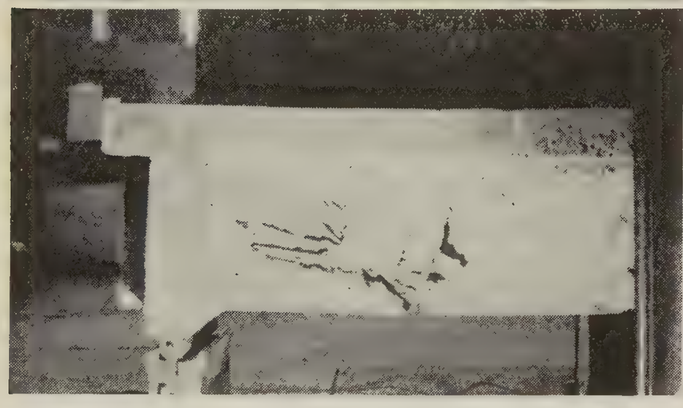


FIG. 8—BEAM NO. 3. CONCRETE CRUSHED UNDER HOOKS AND BENDS
FIG. 10—BEAM NO. 4. BENT BARS BURST THROUGH SURFACE OF BEAM

ing of the concrete occurring at a load of 75,000 lb.—approximately half the load required to bring about the failure of beam No. 1. Fig. 8 shows the condition of the beam after the maximum load had been reached. The hook on the end of the straight bar crushed the concrete when the stress at G.L. 28 (37 on the opposite side of the beam) was less than 30,000 p.s.i., and the middle bent bar split out the side of the beam when the stress at G.L. 16 was 41,000 p.s.i. The destructive effect of hooks and bends, particularly in bars of larger diameters, is thus forcefully demonstrated. It will be noticed too, that stress at G.L. 18 was consistently higher than that at G.L. 16, showing that the bend in the large, stiff bar was acting as an anchor. Slip was apparent both at points A and B at comparatively low loads, and was plainly visible to the naked eye when failure occurred. The maximum unit shear developed in beam No. 3 was 545 p.s.i.

Beam No. 4. Beam No. 1 having proved capable of carrying such heavy loads, and failure having been caused by the crushing of the concrete under the hooks, it seemed advisable to investigate the possible increase in carrying capacity due to the

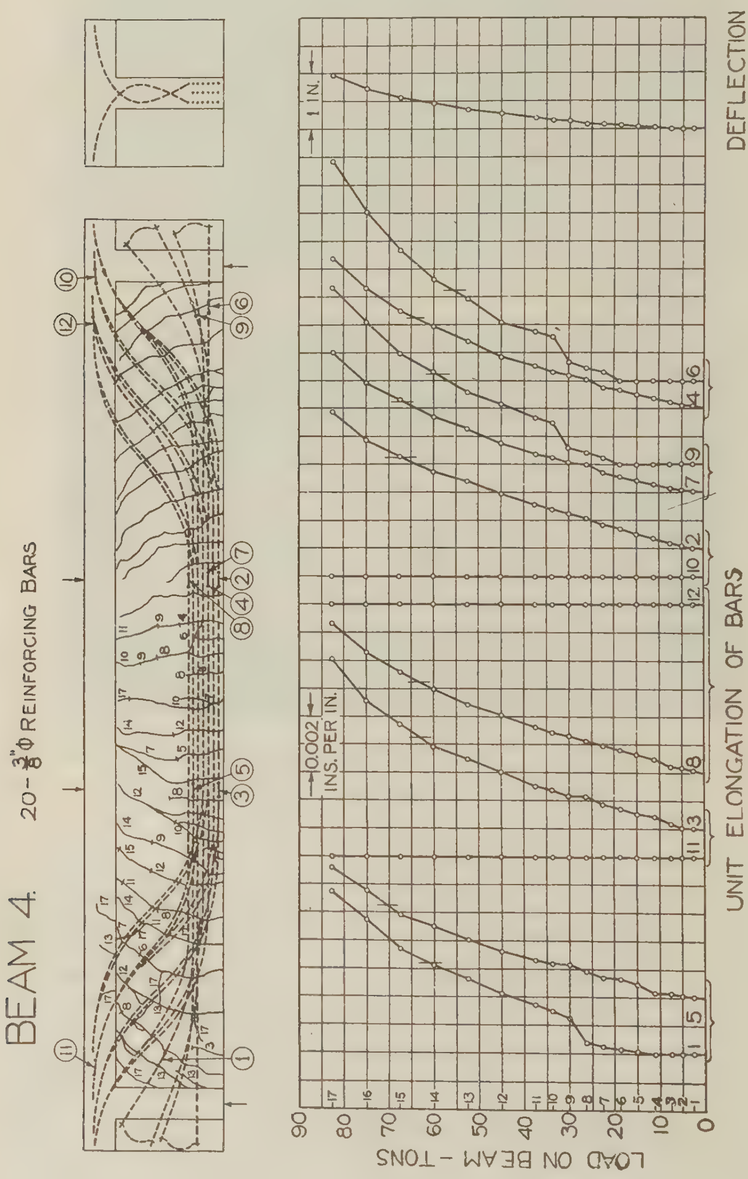

BEAM 4
20-3/8"φ REINFORCING BARS
LOAD ON BEAM - TONS
UNIT ELONGATION OF BARS
DEFLECTION
0.002 INS. PER IN.
1 IN.

FIG. 9

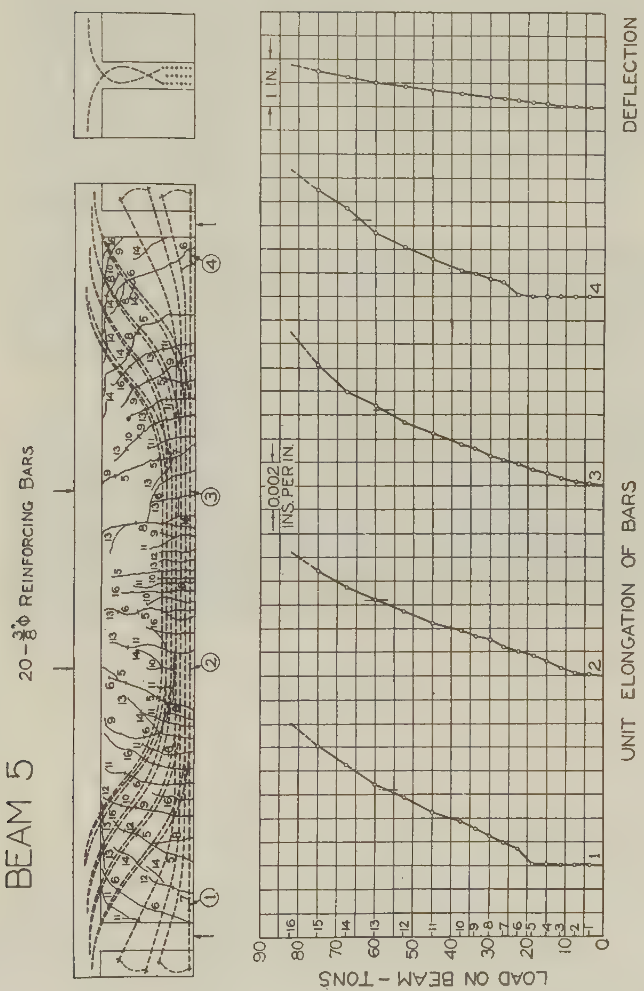

BEAM 5
20 - 3/8" Φ REINFORCING BARS
0.002 INS. PER IN.
1 IN.
LOAD ON BEAM - TONS
UNIT ELONGATION OF BARS
DEFLECTION

FIG. 11

FIG. 12—BEAM NO. 5 AT FAILURE IN TESTING MACHINE
FIG. 13—BEAM NO. 5. DIAGONAL CRACK BETWEEN LOADS

use of more effective anchors and steel having a higher elastic limit. Hence rolled deformed steel bars having an elastic limit of 96,000 p.s.i. were secured, and anchors of the Scott type were used. With these two exceptions, beam No. 4 corresponds exactly to beam No. 1. Fig. 1 shows the reinforcement for beam No. 4 in the forms, the anchors being plainly visible. In Fig. 9 the crack pattern is once more seen to approximate that of the homogeneous beam. No stress was discernible at G.L. 10, 11, and 12, which indicates the effectiveness of the bends employed, and suggests that in the flanges of simple beams it may not be necessary to use Scott type anchors. Elsewhere, high stresses were developed. The deformations everywhere corresponded to unit stresses much higher than the elastic limit of the material used. The load carried, 165,000 lb., developed a unit shearing stress of 1195 p.s.i. Failure finally occurred when one of the bent bars close to the surface burst out the side of the beam, as shown in Fig. 10.

Beam No. 5. As stated elsewhere, beam No. 5 was just like beam No. 4 except that interlacing of the bars was eliminated. The reinforcement, crack pattern, and test data are given in Fig. 11. The same regular distribution of cracks is evident, and

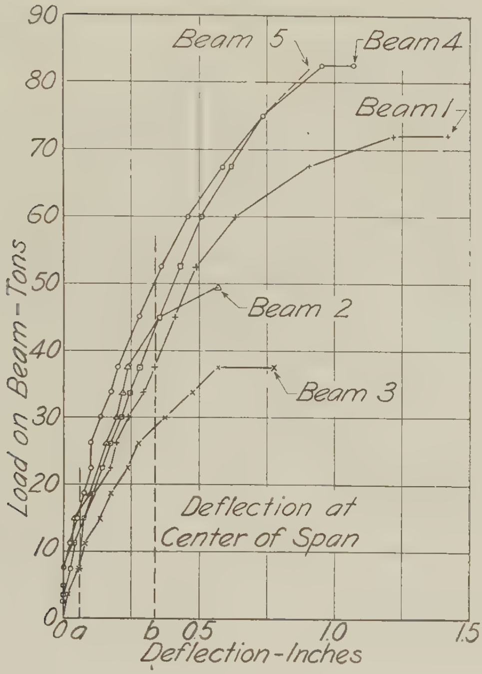


FIG. 14

the same maximum load was attained. At its second application, however, the beam literally exploded. One flange between load points flew off. Immediately a large crack crossed the beam, making an angle of almost exactly 45 deg. when viewed either in plan or in elevation. Since the crack formed between loads, in a region of zero external shear, and since the edges of the crack were badly crushed, it is evident that failure was due to the shear accompanying compression. The failure is shown in Fig. 12, when the beam was still in the testing machine, and in Fig. 13, after removal.

BOND

A study of the deformation diagrams yields some interesting facts about bond. In beam No. 1, for example, the stresses at all gage lines differ but little from one another. The maximum difference occurs at load No. 7 between G.L. 58 and 60, 17 in. apart on the same bar. This difference of about 15,000 p.s.i., divided by the surface of the bar between gage lines indicates an average bond of only 143 p.s.i. The difference grows less as the load increases, indicating a breakdown of bond between the two points. At the maximum load the nominal bond, computed by the formula $u = \frac{V}{\Sigma ojd}$, is found to be 3980 p.s.i., and even if the four bars not bent into the flanges be

included in evaluating Σo the computed nominal bond still reaches the fantastic figure of 995 p.s.i.

Similar results may be observed in all the beams, whether reinforced with rolled or drawn bars, deformed or plain. It is obvious that while bond may be depended upon at low stresses, it is not capable of developing those high steel stresses which would prove so economical with the high elastic limit steels now available. Anchorage is therefore necessary. The best practical form of anchor so far in general use is the semi-circular hook. In beam No. 3 such hooks developed a stress of 28,000 p.s.i. in ¾-in. round bars, (See G.L. 28) while in beam No. 2 at G.L. 37 a stress of 30,000 p.s.i. was attained in ⅜-in. round bars. In beam No. 1, where the steel was so arranged as to make splitting of the concrete more difficult, G.L. 56 and 60 show stresses of 60,000 p.s.i. and 70,000 p.s.i. respectively, although all three beams failed owing to the splitting action of the hooks. At these stresses, however, the bars in beam No. 1 slipped more than ⅜-in., the slip being reflected in the curvature of the deflection diagram at high loads—a curvature scarcely noticeable in beams No. 4 and 5. The Scott anchors in beams No. 4 and 5 developed steel stresses far in excess of the 96,000 p.s.i. elastic limit, and there was no evidence of distress in the concrete due to their use.

DEFLECTION

The deflection diagrams have all been assembled for comparison in Fig. 14. It is proposed to introduce into the forthcoming American Concrete Institute specifications for buildings the following formula for maximum deflection under test load: $\Delta = \frac{0.001L^2}{t}$. For the beams under discussion this would give $\Delta = 0.06$ in., and this deflection is indicated as point a on the lower margin of Fig. 14. The corresponding loads show a factor of safety of approximately 3 for beam No. 2 and of 4 for beam No. 3, values, quite satisfactory for the unit working stresses proposed. For beams No. 1, 4, and 5 the factor of safety is approximately 5½, a rather luxurious figure.

The usual building code, when referring to structural steel, limits deflection to $\frac{1}{360}$ of the span. This deflection for the beams in question is plotted at point b Fig. 14. It is obvious at once that the load required to produce this deflection is dangerously close to the failure load in beams No. 2 and 3, but not in beams No. 4 and 5. In beams reinforced as were these latter two, a factor of safety of 2½ would

still be obtained when the load was increased sufficiently to cause a deflection of $\frac{0.0027L^2}{t}$, provided that time yield under long continued load did not seriously modify the conclusions. This is a point that will bear further investigation.

RECOVERY

The limitations on the scope of the program did not permit as detailed observations on the recovery from deflection as might have been desirable. The following readings, however, are significant. Beam No. 1 carried a load of 120,000 lb., i.e. 83⅓ per cent of the ultimate load, for 3 days, during which time the deflection increased from 0.63 to 0.71 in. One minute after the removal of the load the deflection had decreased to 0.28 in. and the recovery would have been greater if time had permitted. Expressed differently, 61 per cent of the maximum deflection disappeared in one minute, leaving a residual deflection of 39 per cent of the maximum.

A load of 120,000 lb. was left on beam No. 4 for 45½ hours, the deflection increasing from 0.48 in. to 0.50 in. This is much less than in beam No. 1, probably owing to the fact that bar slip had been prevented. Later a load of 135,000 lb. was left on the beam for 20 hours, and the deflection now increased to 0.58 in. One minute after the removmal of the load only 0.1 in., i.e. 17 per cent of the maximu was discernible.

On beam No. 5 a load of 150,000 lbs., i.e. 91 per cent of the maximum, was left for 16 hours, with a deflection of 0.73 in. One minute after the removal of the load the deflection had decreased to 0.18 in.—only 25 per cent of the maximum. On the second application of the next load the beam failed.

Fig. 3, 4, 6, and 8 show clearly the slip of bars at failure in beams No. 1, 2, and 3. In beam No. 3 the slip was measured and is recorded in Fig. 7. Later examination showed that this slip had polished the concrete under the bends and hooks of beams No. 2 and No. 3 and under the hooks of beam No. 1 Obviously, slip contributed largely to the deflection of the first three beams. In such beams as these the clause which limits the load to that load, applied for 24 hours, from which the beam will recover 75 per cent of the deflection, is a first class safeguard against excessive slip. But this clause would be dangerous if applied to beams No. 4 or 5, for both these "one hoss shays" recovered 75 per cent of the deflection caused by loads nearly as great as those causing failure. This surely proves how difficult a task it is to write a specification which will cover all cases, how easy

it is to write a clause which applies only to the particular picture in the writer's mind at the time of writing, and how the possibility of future development should be kept continually in view.

In conclusion it may be stated that with proper anchorage it is possible to develop the elastic limit of any steel that is likely to come into commercial use, and with proper bending and anchorage the question of bond is eliminated and shear stresses are governed only by the strength of the concrete in diagonal compression. We have learned how to control the quality of concrete and are discovering the great benefits of vibrating. Today the opportunities for using in design a whole new fund of knowledge are greater than ever before.

For such discussion of this paper as may develop, readers are referred to the JOURNAL *for November-December 1934 (Proceedings Vol. 31). Discussions should be available to the Secretary by October 1, 1934.*

Tendencies in Canadian Railway Bridge Design—Recent Work on the Canadian National Railways*

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MEMBER AMERICAN CONCRETE INSTITUTE

In the last twelve or thirteen years on the Central Region of the Canadian National Railways, there has been a steady development in the design of railway and highway structures—so gradual that it has only lately had the deserved attention of the engineering world. Recently, changes from conventional designing have become so revolutionary as to win considerable attention from the engineering press. No engineering society, however, has reviewed the whole development of bridge structures.

With such a wealth of available material it will be possible only to touch the high spots and as this is a concrete convention, I will deal for the most part with concrete structures.

As Mr. Sedgwick[1] deals with the recent practice in Ontario with regard to structures carrying highways over railways, I shall consider that class of bridges carrying railway traffic over highways, streets, valleys or streams. There is great interest now in both the United States and Canada in grade crossing eliminations, on which millions of dollars have been and will be spent as an unemployment relief measure. For this reason, economical subway construction is very desirable.

In the last 15 years the Bridge Department of the Canadian National Railways Central Region, under C. P. Disney, Bridge Engineer, has been designing and building many of these subways and designed many different types. By process of gradual elimination the department feels that seven types of structures to be discussed meet practically every condition of subway design.

The sequence of presentation of these types is no indication of their merits; each design is adapted to a different condition.

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*Presented at the 30th Annual Convention American Concrete Institute, Toronto, Feb. 20-22, 1934.
†Consulting Engineer, Canada Cement Co. Limited, Montreal.
[1]Arthur Sedgwick, Bridge Engineer, Ontario Dept. of Highways: "Rigid Frame Highway Bridges in Ontario." See p. 479.

FIG. 1 AND 2—SUBWAY NEAR ST. LAMBERT. SIMPLE SLAB CAST IN PLACE. CLEAR SPAN 42 FT. NO BALLAST, NO TIES. RAIL HELD BY CLIPS AND ANCHOR BOLTS. ONE INCH OAK BOARD UNDER RAIL

TYPE 1

Type 1 is illustrated by a structure near St. Lambert, Que. (Fig. 1 and 2)—A simple beam, poured-in-place, without ties; suitable for use where railway traffic need not be maintained and where the clear span does not exceed 60 ft. The simple beam, overall length is 57 ft.

11¾ in. on the skew, giving a clear unobstructed roadway of 42 ft. at right angles to the face of the two gravity abutments on which it rests. This structure was built in 1931.

As the rails are fastened directly to the 4-ft. thick slab (there being no ties nor ballast) this is the distance from base of the rail to underside of structure—an important dimension in subway design. The smaller this dimension the more economical the structure. One dimension is always fixed—the distance from the top of the finished roadway to the underside of the structure must not be less than 14 ft. Thus it is usually important to keep distance from the base of rail to underside of structure as small as possible, in order to cut the cost of drainage, sewers, property damages, etc. to a minimum. Elimination of ties and ballast saves at one stroke, from 15 to 18 in. and at the same time a highly desirable track structure has been obtained.

Some of these track structures have been in use for more than six years and have given perfect satisfaction. When they were introduced many difficulties were predicted—such as severe shock when the train came off the approach grade onto the structure of the bridge. The fact that any engine coming off any grade onto the backwall of any abutment on any bridge created exactly the same condition as existed on these structures seemed not to be considered. In six years of operation, no shock in coming off the approach grade onto these structures has been experienced.

The riding qualities of this track are easily the best on any portion of the Canadian National Railways tracks, in that there are no low joints, there is no bending or spreading of the rails; a perfect track is always provided. Moreover, the track is unusually quiet, due partly to the oak strip and partly to the fact that it is on a heavy mass of reinforced concrete. Where these subways have been built in cities, it has been remarked how noiseless they are.

A further important consideration is absence of maintenance—no timber ties to be renewed; no ballast to be tamped and packed around the ties, there is no accumulation of dirt and filth on the track, which is especially important in terminals and cities. The track can be kept spotlessly clean with a hose or broom and always presents a neat appearance.

The saving of 14 in. of headroom by the elimination of ballast in some instances has an important economic aspect. In one instance, every inch of height saved in the depth of the subway meant a saving of $100,000, in the grading of an adjoining yard which had hundreds

of tracks. Therefore, a saving of 15 inches in headroom by the adoption of this type of structure would have effected a saving of $1,500,000. It was not saved, as the old type of ballasted track was used.

In construction of the work shown in Fig. 1 and 2, the slab was poured in place, that is falsework was erected and forms built and the slab poured right on the abutment. This was possible because the line was not under traffic. If the line had been under traffic, the slabs would have been pre-cast and dropped into place between trains, which is only a matter of a few minutes. This feature of speedy erection will be dealt with as another type of structure. Such structures have nothing on them that should ever need renewal, except the rails.

TYPE 2

Type 2 is a simple beam pre-cast, no ties—suitable for use where railway traffic has to be maintained and where the clear span does not exceed 60 ft. Such a structure is identical to Type 1, except that instead of its being poured in place the slabs are pre-cast and lifted into place between trains by two locomotive cranes. Everything said regarding subway structures of type No. 1, applies equally to subways of Type 2. While it is not illustrated, pictures of Type 5, considered later, illustrate this pre-cast construction.

TYPE 3

In Type 3 the span is continuous over centre support, cast in place, with concrete ties—suitable for use where railway traffic does not have to be maintained during construction of the bridge, where a centre pier is permitted, and where the clear span on either side of the pier does not exceed 50 ft., that is, where the total length of the continuous slab does not exceed 100 ft.

An example of this type is the Richmond Street subway, London, Ont., (Fig. 3 and 4) built in 1931, designed for Cooper's E.60 loading. There is also a roadway designed for 20-ton trucks.

The extreme length of slab overall in this instance is 73 ft. 9 in.; depth from base of rail to underside of slab 3 ft. The structure carries 13 railway tracks, one 24 ft. 3 in. roadway, one 3 ft. 9 in. sidewalk on the north side, and one 7 ft. 11½ in. sidewalk on the south side.

After the concrete slab was poured and set up, pre-cast concrete ties 5 in. deep, with the necessary rail fittings pre-cast into them were placed on the finished surface of the concrete slab, then the spaces in between the concrete ties were filled in with concrete, making a continuous flat surface over the whole track structure. The reason for

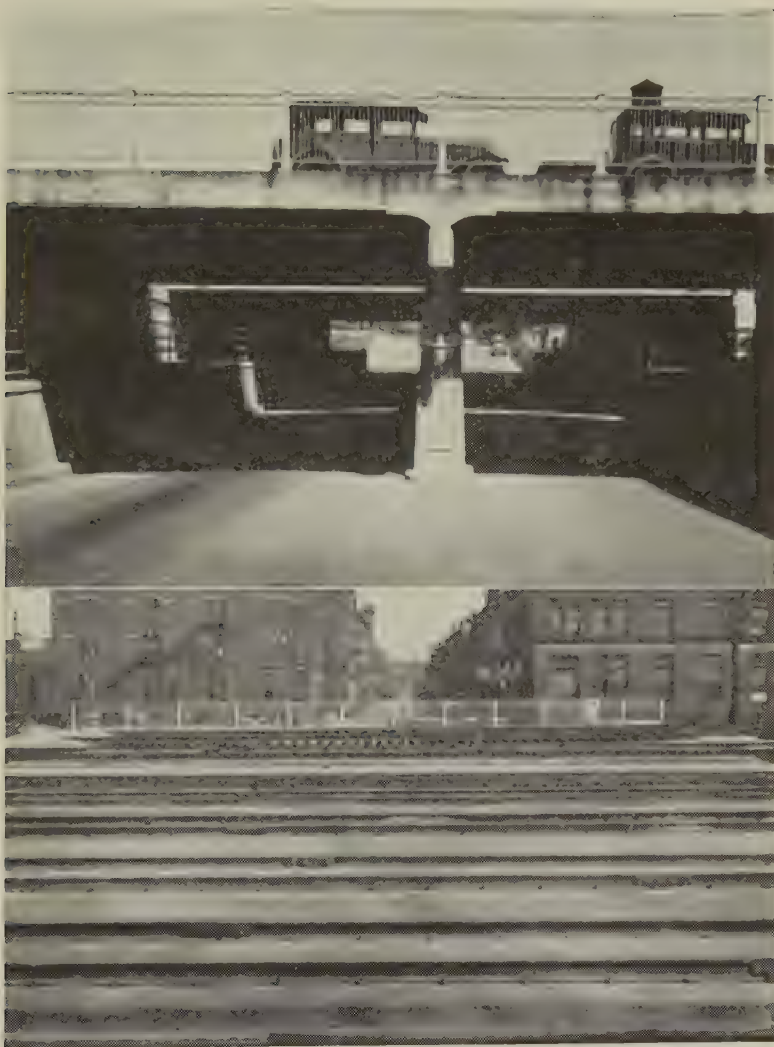

FIG. 3 AND 4—RICHMOND ST. SUBWAY, LONDON, ONT. CONTINUOUS OVER CENTER SUPPORT, CAST IN PLACE. NO BALLAST, BUT WITH CONCRETE EMBEDDED CONCRETE TIES. TWO BRIDGES—ONE FOR STREET TRAFFIC, ONE FOR 13 RAILWAY TRACKS

using concrete ties on this structure was in the fact that it is in the middle of a yard terminal, where there are many switches, crossovers, curved tracks, etc.

It was also necessary to have some freedom here to line up the track

after the subway was built as the track layout was not finally completed before the subway was actually constructed. By using these light concrete ties it was a simple matter to line the tracks up in any way that was necessary. If concrete ties had not been used here, the distance from the base of rail to underside of track would have been but 27 in. This is very remarkable and at least a couple of feet less than on any other possible type of design, and the saving at this location was considerable, as it was in the heart of the city where sewers, property damages, raising of railway tracks, etc. were factors.

TYPE 4

In Type 4 a pre-cast beam is continuous over a centre support, with no ties—suitable for use where railway traffic has to be maintained during construction and where the clear span does not exceed 45 ft. on either side of the pier, or the total length of the continuous slab does not exceed 90 ft.

An example of this type is the subway at St. George, Ont., built in 1932, carrying a single railway track (Fig. 5 and 6). Its total overall length is 58 ft. 10 in. A second example is the St. Francois Street subway at Sherbrooke, built in 1932, designed for Cooper's E.60 loading, carrying a single railway track; total overall length 53 ft. 2⅝ in. and distance from base of rail to underside of slab was 24⅜ in. On this slab the rail is carried directly on an oak strip embedded in the slab. No ties of any kind were used. A remarkable feature of this slab is that it was erected in 20 minutes between trains. (Fig. 7).

TYPE 5

Type 5 is a rigid frame continuous over centre support, poured-in-place, with concrete ties—suitable for use where a center support is permissible and where the clear span required on either side of the center support does not exceed 60 ft. An example of this type is the St. Clair Avenue subway, on the Newmarket Subdivision, built in 1931, designed for Cooper's E-60 loading, where the extreme length of slab along the track is 111 ft. 9¼ in. The distance from centre line of pier to end of slab is 65 ft. 10⅝ in.; face to face of abutments is 102 ft. 6¼ in. providing a 100-ft. clear width of street. (Fig. 8 and 9).

It carries a double track railway on concrete ties, with a concrete platform 8 ft. wide on the east side and a concrete sidewalk 3 ft. wide on the west side. The tracks are on a 2 deg. 30 min. curve, and there is a 5 per cent grade on the bridge. The distance from base of rail to underside of slab is 4 ft. 4 in., which is very remarkable. This is one of the finest looking structures on grade separation work and it is also probably the most economical structure of its size ever built.

FIG. 5 AND 6—SUBWAYS AT ST. GEORGE AND AT ST. FRANCOIS ST., SHERBROOKE

Precast slabs continuous over center support. No ballast, no ties. Top to bottom the pictures show: Leading slabs; removing falsework after erection under traffic. This double bent carried one end of one slab alongside old pile trestle that was taken out. Erection done "between trains."

FIG. 7—COMPLETED STRUCTURE AT SHERBROOKE (SEE PREVIOUS PAGE)

The construction was very simple and very rapid, the whole structure proper being poured continuously for 72 hours. The placing of steel reinforcing took about five days. There is no maintenance or upkeep and there is no shock whatever coming off the approach grade onto the structure. There is little noise from the passage of a train—even when standing under it.

It is a safe statement to make that this is the only structure of this kind ever built, that is, a fixed frame reinforced concrete structure continuous over but not fastened to the centre support, giving a complete reversal of stresses over the centre support and involving a set of equations different from any ever used before.

TYPE 6

Type 6 is of rigid frame, poured-in-place, without any ties or ballast—suitable for use anywhere where a clear span is desired, that is, where no centre support is permissible and where the clear span required does not exceed 100 ft.

This is the ideal structure where a clear span is over 60 ft. and does not exceed 100 ft. And, owing to its beautiful appearance it could be used to advantage and at a very little extra cost in many locations where we would otherwise use a simple slab design.

An example of this type has just been completed at Vaudreuil, Que. (Fig. 10, 11 and 12) where a double track structure carrying what is probably the heaviest and fastest traffic anywhere in America, was built and put into operation in the last six months.

FIG. 8 AND 9—ST. CLAIR AVE., TORONTO

Rigid frame, freely supported at center. A very unusual design. No ballast, but with concrete embedded concrete ties. Splendid appearance. Clear span 50 ft. each side of center pier.

The clear span face to face of abutment is 72 ft. $6\frac{3}{4}$ in. The thickness of the slab at the centre is only 3 ft. 9 in. The structure was built under traffic. For this and other reasons it was built as two separate units, that is, one unit was completed first, the traffic then diverted over this completed unit, and the unit carrying the other track was then placed. There is absolutely no connection between the two units, except a strip of sheet lead placed horizontally continuously through the structure to make a waterproof joint.

As the structure is on a skew, a heavy torsion stress was produced. If the structure had been built all in one piece it can readily be seen

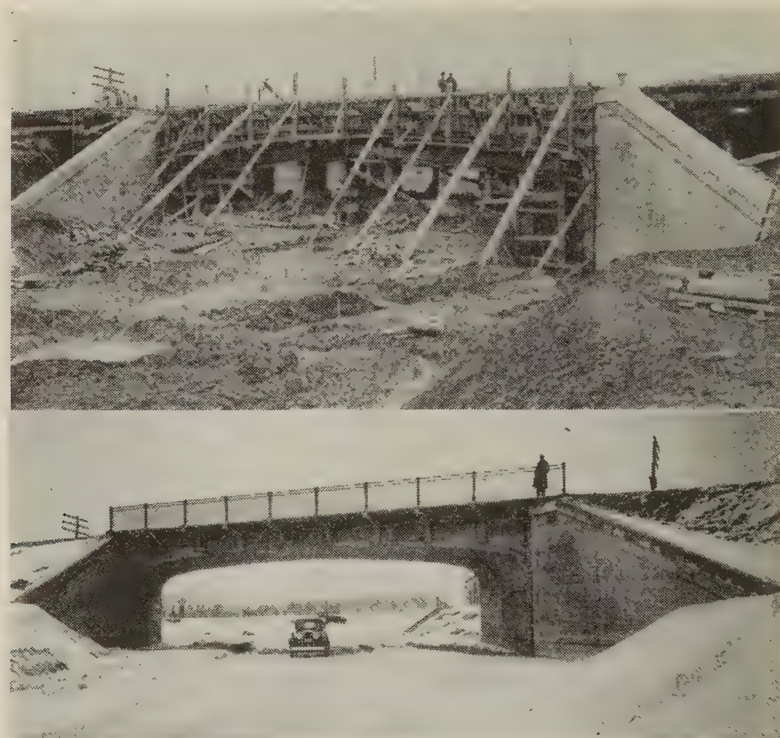

FIG. 10 AND 11—VAUDRIEUL SUBWAY

The "Daddy of Them All." 72 ft. 6¼ in. span. Built in three weeks, under heavy high speed traffic in one month, and subjected to 45 degrees F. below zero in three months. No ballast, no ties, no deflection, no noise.

that the torsion would have been doubled, as when two heavy trains met on the bridge travelling in opposite directions, it would have a tendency to twist and distort twice as much as is now the case with two portions of the bridge kept separate.

A structure of this type if built on a grade separation project not under traffic could, without doubt, be completely built in three weeks by a contractor who had an efficient plant and who knew his business. This is a very remarkable preformance and no other type of structure of this kind, size and design could be built in twice the time. To the best of our knowledge this is the only real through rigid frame structure carrying railway traffic that has ever been built. It is certainly the only one that has ever been built without ties and ballast.

It was designed for a very heavy railway loading, that is, Cooper's E.60, and within one month after its erection has been subjected to

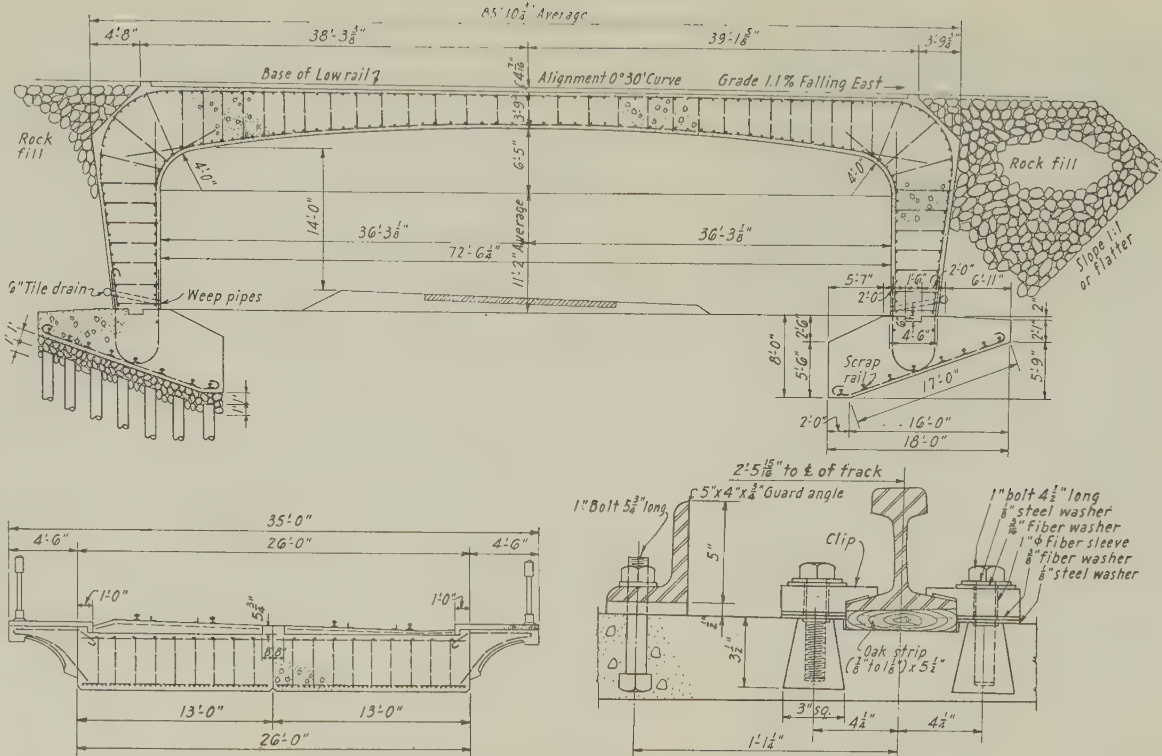

FIG. 12—FIXED FRAME BRIDGE, VAUDRIEUL, QUEBEC

A double track structure on a skew built under traffic—one-half at a time. No ballast, no ties. (Top) Longitudinal section approximately to scale. Note unusual foundation design—piles separated from footing they support. (Left, below) Cross section at center of span. (Right) Details of rail fastening and support.

most severe temperature stresses. Within the last two months it has repeatedly been subjected, for days at a time, to a temperature that has been well under 45° below zero, for periods up to five days. Keeping in mind the fact that the concrete was only about 28 days old when this occurred, one can readily see that this structure has been subjected to the heaviest stresses it will be called upon to carry. Needless to say the structure has come through this test, apparently 100 per cent perfect.

In building this structure, every possible care was taken to secure perfect concrete. Vibrators were used and high early strength cement. Every care was taken in placing the concrete to get a continuous and rapid pour. The cement, sand, stone and water were all carefully inspected and analyzed in the laboratory before being used. The concrete was designed for a compressive strength of 3,000 p.s.i. at seven days. The actual strength obtained varied from a minimum of 3,200 to a maximum of 3,950 p.s.i., the average being 3,605. After two days some of the cylinders were broken and a strength of 2,800 lbs. was obtained.

Should there be any question regarding the type of foundations for a structure of this kind, it is to be noted that the foundation in this bridge consists of what is known as Quebec blue clay, which, when dry, has a good bearing capacity, but when wet is a quick sand. Consequently, the foundations were piled, the piles being driven vertically but cut off on a sloping plane at right angles to the resultant of all stresses throughout the structure. Unique is the the fact that tops of piles were cut off one foot below the bottom of the concrete foundations; the clay was then excavated to one foot below the cut-off; a 2-foot layer of crushed stone was then placed and tamped around the piles to a distance of one foot above the top of the piles; on this stone mat the concrete foundations were placed.

It is Mr. Disney's contention that all pile foundations should be built this way. He contends that if you place the concrete around and in contact with the heads of your piles, the piles then have to take all the load, the earth carrying practically nothing. This can be proven when demolishing an old structure—the earth is not even in contact with the bottom of the structure. However, if you do not put your concrete in contact with your piles, but put a layer of crushed stone over them, you then have a solidified area with increased bearing capacity sufficient to carry the loads.

It is not difficult to determine the bearing capacity of an area of this kind; you simply drive piles until you cannot get any more in.

Fig. 13—Breslau subway

Steel girders and floor beams, but with reinforced concrete stringers carrying rails. No ballast, no ties.

In some cases it may be necessary to pile outside the actual area over which the concrete rests in order to obtain a thoroughly safe condition.

TYPE 7

Structural steel in a through place girder span, with steel floor beams between the main girders carrying reinforced concrete T-section slab, and no ties or ballast are features of Type 7.

This type of structure is for use when structural steel is used in either grade separation work or certain steel railway bridge work.

The conventional type of ballasted steel subway or steel railway bridge, still being built throughout America, consists of main steel girders with steel floor beams between the girders and then running at right angles to these floor beams are numerous I-beams spaced from 12 to 15 in. and usually 15 to 20 in. deep. This steel structure is designed to carry all the loads passing over it.

When the steel structure is completely erected, then a solid concrete filling is placed between and around the I-beam stringers and floor beams and is carried well up on the side of the girders. On top of this is then placed an expensive waterproofing fabric, and other protection, usually reinforced concrete. When this waterproofing protection has been completed and set up, then about 14 to 18 in. of stone ballast

with timber ties embedded in the ballast is put in place, on which the rail is set.

The cost of this extra material is very high and usually increases the cost of the structure about 40 per cent, because not only is it waste material, but it adds to the dead load of the steel structure, by requiring heavier floor beams, stringers and main girders than otherwise necessary.

The Breslau subway (Fig. 13) is designed as economically as possible. There are the usual plate girder and floor spans, but there the similarity stops. Hundreds of heavy steel stringers and the two or more feet of solid concrete, serving as only a filler, are entirely eliminated; the 14 to 18 in. rock ballast is eliminated as well as the creosoted timber railway ties, and for all this is substitutèd a simple T-section reinforced concrete slab having about 10 per cent of the concrete in it that is wasted in the other type of design.

This concrete slab carries the whole load successfully and makes use (as all first class designing should) of all engineering materials in the structure. It gives a perfect track structure on which there is no maintenance; it gives a minimum of headroom and it is Mr. Disney's contention that all steel railway bridges should in future have this type of track structure, as this will eliminate all timber ties with their maintenance costs. The replacement of timber ties on bridges of the open deck type, also of the ballasted deck type on a railway is one of the most serious maintenance charges, and over a period of years it amounts to millions of dollars. Mr. Disney has no hesitation in stating that no more steel railway bridges with timber decks should be built and that no more railway bridges with ballasted decks should be built.

The initial cost of the type of deck illustrated here is just about the same as the initial cost of creosoted timber decks and is about 40 per cent less than the initial cost of the ordinary type of ballasted deck consisting of numerous steel stringers, solidly filled in with concrete, upon which the ballast and ties are placed.

It is readily seen why grade separation work is expensive and why more of it is not done.

For such discussion of this paper as may develop, readers are referred to the JOURNAL *for November-December 1934 (Proceedings Vol. 31). Discussions should be available to the Secretary by October 1, 1934.*

Rigid Frame Highway Bridges in Ontario*

BY ARTHUR SEDGWICK†

Old Ontario is, in general, a more or less flat plain with many small streams often flowing between shallow banks. During floods and especially spring break-ups, the river channels are taxed to their full capacity. It is therefore most important in bridging these streams to see that no obstruction is offered to the free flow of the water. In many instances the road approaches to the stream must be raised to afford sufficient depth of waterway beneath the bridge. To raise the road and bridge floor level abruptly above the ajoining land on either side of the stream is to perpetuate an eyesore in what may already be a somewhat uninteresting landscape. In such locations the bridge if possible should form a part of the level road and be as inconspicuous as possible.

Ontario's trunk highways usually follow the original County roads on which are many old structures which must be replaced by new structures with much wider roadways and capable of carrying much heavier loads. Under previous practice, these modern structures would require a much deeper floor system than the structures they replace. This in turn would necessitate raising the road level to provide sufficient waterway.

In most instances the old bridges are of the "through" type. For several reasons a deck bridge is more desirable for modern roads and traffic than the through type. A through bridge especially of steel, is in danger of being damaged by collision. It also offers some obstruction to vision. A deck bridge can be widened in the future without sacrificing the existing bridge or disturbing traffic. A deck bridge presents a more pleasing and substantial appearance and can be made to appear as a link in the modern highway instead of offering an obstruction and menace to traffic.

For these reasons we welcomed the introduction of the rigid frame structure.

For highway bridges carrying H 20 loading, the depths of the deck

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of a rigid frame vary from about 1/40 of the clear span at the center to 1/15 of the span at the abutments. On the other hand, the corresponding depth for a girder bridge would be uniformly 1/12 of the span. Taking a sixty foot span for example, there would be a saving in height at the abutments of exactly 1 ft. and at the center, a saving of 3 ft. 6 in. The quantity of concrete and reinforcing steel in the deck is practically the same for either type of structure. For ordinary heights of abutments there will be considerably less concrete in the rigid frame abutments than for gravity walls of the same height but there will be much more steel reinforcement in the former type. There is somewhat less of both material and labor in the form-work for the rigid frame structure than for a beam and slab structure. On the whole, therefore, we find that up to spans of 60 ft. the rigid frame bridge can be built for somewhat less money than can a bow string arch or a beam and slab design.

Commencing in 1931 we built four such structures in Essex County which is particularly flat country. The results in economy and simplicity of construction were entirely satisfactory. These structures varied in length from single spans of 25 to 58 ft. and twin spans of 45 ft. each.

In the Autumn of 1931 we had the problem of designing a fixed bridge over the Scugog River to replace one steel fixed span and one timber swing span on the King's Highway No. 7 and operated by the Department of Railways and Canals, Ottawa. The Department of Railways and Canals had previously prepared plans and estimates for two 80-ft. steel plate girder spans with a 24-ft. roadway and one 4-ft. sidewalk. This design provided for a 30 ft. boat channel on either side of a central pier. To provide a ten foot vertical clearance it was necessary to use a 2½ per cent grade from both ends of the bridge up to the center pier. This latter feature was objectionable to vehicular traffic. An alternative design was therefore prepared using three 50-ft. rigid frame concrete spans with a 35 ft. channel through the center span with a minimum vertical clearance of 10 ft. The roadway was increased to 30 ft. and the sidewalk to 5 ft. In spite of the wider roadway this design permitted the floor level to be lowered about one foot at the center to replace the proposed 2½ per cent grade up to the center, with a smooth longitudinal camber of 6 in. in a length of 160 ft. The rigid frame design was adopted and tenders received on this design. The contract price for this design with 30 ft. roadway and 5 ft. sidewalk about equaled the estimated cost for the plate girder spans with 24 ft. roadway and 4 ft. sidewalk. The footings for the rigid frame structure were all carried to bed rock

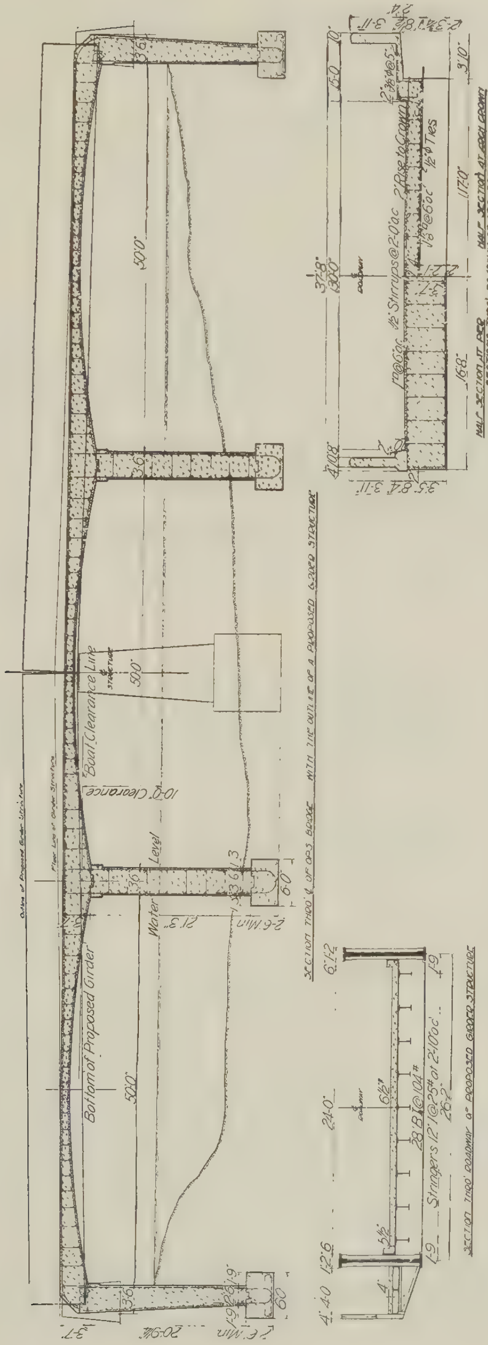

FIG. 1— EXAMPLE OF RECENT ONTARIO HIGHWAY BRIDGE DESIGN—AN ALTERNATE GIRDER CONSTRUCTION IS OUTLINED

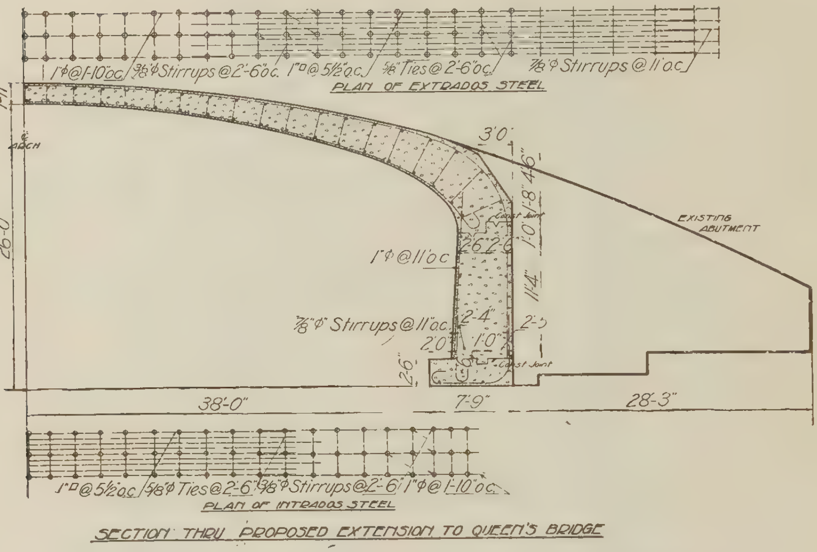

FIG. 2. SECTION OF PROPOSED BRIDGE EXTENSION

which had not been contemplated when preparing the estimate for the steel design. We consider the wider roadway and easier grades obtained for the same money as estimated for the previous design more than justified our adoption of the rigid frame structure for this location.

In 1932 plans were prepared for widening the roadway for an existing 80 ft. span concrete arch built some 25 years ago. This was an eliptical arch with a rise of 12 ft. in a span of 80 ft. The skew backs were each made 35 ft. wide. In extending the arch the arch ring was made the same depth and form as the existing structure but the rigid frame principle of design was employed and instead of skewbacks 35 ft. wide being used, a rigid frame vertical leg was designed with a footing only 7 ft. 9 in. wide. This improvement results in a saving of 75 per cent of concrete in the abutments. The saving in excavation is even greater than this. The saving in reinforcement in arch ring and abutments is about 30 per cent,—this being due to the existing arch being uniformly reinforced both in extrados and intrados throughout the length.

The rigid frame design can be shown therefore to afford pronounced economy over any orthodox arch design with fixed skewbacks in earth foundations where the springing line must be kept above high-

water line and the depth of footings carried down below any danger of scour.

The rigid frame has distinct advantages in economy over freely supported spans in cases where a moderate span length but high abutments are required. In such cases a marked saving in the cost of the abutment can be effected. For instance, let us assume a span of 60 ft. and a clear height from bottom of footings to underside of deck of 20 ft. For a freely supported beam span and gravity abutments the thickness of walls at the bottom would be 33 per cent of the total height. Such abutments would therefore have a width of 8 ft. 8 in. at the bottom of footings and at the bridge seat about 4 ft. A rigid frame structure on the other hand would have a width of wall at the top of approximately 4 ft. tapering to 2 ft. at the top of footings and with footings probably 5 ft. wide and 2 ft. 6 in. deep. The saving in the quantity of concrete would be approximately 50 per cent. This saving in concrete would be partly offset, however, by the addition of the necessary steel reinforcement. The same saving might be accomplished by using a counterfort type of wall for the beam and slab construction but in that case the cost of formwork would be increased.

In 1932-33 we built twin and triple spans up to 60 ft. in length each, in each case believing we were securing the most economical and most attractive structure for each location. In using this type of construction it is necessary of course to be sure of an unyielding type of foundation. For multiple spans the foundations must be beyond suspicion as any yielding of one footing with respect to the others would entirely upset the conditions of stress, fundamental to the type of design.

With a single span structure we believe some movement of the footings could take place without ruining the structure. In this case we may reasonably assume that if one footing settles in yielding ground it would also tend to move forward due to the earth pressure behind the wall. The result in such a case would merely be a rotation of the whole structure about the other footing. The fundamental assumptions of design would therefore not be violated. At the same time we would not care to recommend this type of structure to be used in treacherous ground.

We have in the last three years built some 20 rigid frame bridges and it is interesting to note that in every instance we have replaced an old steel through span structure with an attractive and durable

deck span at a minimum cost and one which appears to belong to and form a part of the highway or town street instead of being a necessary obstacle thereon.

For such discussion of this paper as may develop, readers are referred to the JOURNAL *for November-December 1934 (Proceedings Vol. 31). Discussions should be available to the Secretary by October 1, 1934.*

Cement Investigations for Boulder Dam with the Results up to the Age of One Year*

BY RAYMOND E. DAVIS, R. W. CARLSON, G. E. TROXELL AND J. W. KELLY†
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THIS paper is supplementary to a paper entitled "Cement Investigations for the Hoover Dam," presented at the 29th Annual Convention of the Institute, 1933.[1] It presents the results of tests up to the age of 1 year for heat of hydration and compressive strength, for the cements and conditions included in the previous paper. Further, it discusses the results of volume-change and durability tests.

The investigations were principally in the Engineering Materials Laboratory of the University of California, to select a cement having most favorable chemical and physical properties for the construction of Boulder Dam. They were conducted under a cooperative agreement between the University of California and the United States Bureau of Reclamation, with the additional cooperation of the California Portland Cement Co., Monolith Portland Cement Co., Portland Cement Association, Riverside Cement Co., Southwestern Portland Cement Co., and Yosemite Portland Cement Corp. The object of the tests was to determine the influence of chemical composition, fineness of grinding and method of manufacture of cement upon heat of hydration, strength, volume changes, and durability of mortars and concretes.

Reference is made to the previous paper for descriptions of the program of investigation, the testing apparatus and methods, the nature of the potential compounds in portland cement, and methods of measuring and designating the fineness of cements. Following are selected test results.

HEAT OF HYDRATION OF CEMENT

In Table 1 are presented the maximum, minimum, and average values of heat of hydration for 20 of the 93 cements employed in the

*Presented at the 30th Annual Convention American Concrete Institute, Toronto, Feb. 20-22, 1934.
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[1]JOURNAL Amer. Concrete Inst., June 1933 (*Proceedings* Vol. 29), pp. 413-431.

investigation, for ages up to 1 year. It is apparent that large differences in heat of hydration can be produced by altering the chemical composition, and that these differences still exist at the age of 1 year.

TABLE 1—RANGE OF HEAT OF HYDRATION FOR LABORATORY CEMENTS OF SPECIFIC SURFACE 1200 SQ. CM. PER GRAM

Range	Heat of Hydration, Calories per Gram of Cement			
	7 Da.	28 Da.	3 Mo.	1 Yr.
Maximum	117	121	123	125
Minimum	25	39	54	69
Average	79	91	98	102

In Table 2 are data showing the effect of chemical composition upon heat of hydration for a number of selected cements which are representative of high-heat, normal, and low-heat compositions. The corresponding values for a larger number of cements have been subjected to a mathematical analysis to find the most probable contribution to heat of hydration of each of the four principal compounds in cement at each age shown. The calculated values of these contributions are given in Table 3. While the probable error of these calculated values is undoubtedly large, they indicate average trends.

TABLE 2—HEAT OF HYDRATION AND MORTAR COMPRESSIVE STRENGTH OF SELECTED LABORATORY CEMENTS OF SPECIFIC SURFACE 1200 SQ. CM. PER GRAM

Cement No.	Compound Composition, Per Cent				Heat of Hydration, (Mass Curing) cal./g. of Cement				Compressive Strength, p.s.i.							
									Mass Curing				Curing at 70° F.			
	C_3S	C_2S	C_3A	C_4AF	7 Da.	28 Da.	3 Mo.	1 Yr.	7 Da.	28 Da.	3[a] Mo.	1[a] Yr.	7 Da.	28 Da.	3 Mo.	1 Yr.
L-2	68	9	12	9	101	110	112	114	4320	5110	5130	6100	3790	5730	6260	6690
L-1	55	23	11	8	86	99	107	111	4080	5070	5480	6410	2470	4300	5740	6270
L-3	26	52	11	8	72	87	94	98	2150	4550	4770	5660	1180	3380	5120	6390
L-4	12	67	10	9	51	66	78	90	900	2620	3320	4350	600	1660	3260	4770
L-5	51	25	18	3	116	121	122	123	3810	4020	3890	4520	2590	3940	4570	5160
L-6	55	24	0	18	75	87	91	92	4290	5740	5820	6720	2330	3700	5600	6360
L-7	12	70	0	15	29	46	59	69	610	2320	4110	5490	280	1370	3800	5530
L-20	31	49	3	15	56	73	82	84	1900	4010	5130	7180	1360	2610	5020	7030

[a]Mass curing for 28 days, then curing at 70° F. until test.

TABLE 3—EFFECT OF CHEMICAL COMPOSITION AND FINENESS OF CEMENT UPON HEAT OF HYDRATION

Age	Contribution of Each Per Cent of Compound to Heat of Hydration, Calories per Gram of Cement[a]				Effect of Increase in Specific Surface of 100 Sq. Cm. per G., Calories per Gram[b]
	C_3S	C_2S	C_3A	C_4AF	
7 Da.	1.1	0.2	2.4	0.0	+2.0
28 Da.	1.2	0.4	2.3	0.1	+1.8
3 Mo.	1.3	0.6	2.1	0.1	+1.3
1 Yr.	1.3	0.8	2.1	0.0	+0.7

[a]For laboratory cements of specific surface 1200 sq. cm. per gram.
[b]Within range of specific surface 1000 to 1600 sq. cm. per gram.

TABLE 4—HEAT OF HYDRATION AND MORTAR COMPRESSIVE STRENGTH OF CEMENTS OF VARYING FINENESS

Cement No.	Compound Composition, Per Cent				Fineness		Water-Cement Ratio, by Wt.[a]	Heat of Hydration (Mass Curing) Cal. per Gram				Compressive Strength, p.s.i.							
												Mass Curing				Curing at 70° F.			
	C_3S	C_2S	C_3A	C_4AF	% Pass 200-Mesh	Specific Surface, sq.c./g.		7 Da.	28 Da.	3 Mo.	1 Yr.	7 Da.	28 Da.	3 Mo.[b]	1 Yr.[b]	7 Da.	28 Da.	3 Mo.	1 Yr.
	54	20	13	9	75.9	960	0.61	97	106	108	108	3160	3950	3740	4530	1960	3710	4250	4880
30M	57	16	13	9	85.3	1150	0.57	99	110	110	110	3830	3840	4390	4410	2830	3740	4890	4710
	61	14	13	9	92.5	1450	0.55	106	111	111	112	4610	4950	5440	5300	4310	5380	6500	7330
	43	30	6	11	80.8	1100	0.61	63	73	84	96	1600	2840	2970	4360	1120	2240	3020	4320
39E	42	31	8	10	89.6	1240	0.59	66	78	87	96	1920	3510	3290	4650	1450	2970	3550	4800
	44	29	7	11	97.0	1570	0.57	76	90	96	97	3750	5070	5450	5710	2470	4390	5530	6320
	39	36	12	9	75.6	1090	0.63	94	101	102	103	2380	3050	3240	3400	1610	3100	3720	4270
15M	38	37	13	7	85.9	1280	0.60	95	102	104	104	2830	3390	3380	3350	2160	3000	4040	3520
	39	34	11	9	94.2	1640	0.59	95	104	104	104	4100	4800	5130	5120	3450	5000	5630	6230
					78.5	970	0.63	45	58	74	83	490	1540	2210	3720	290	880	2220	4030
L-4-2	11	67	11	8	88.5	1130	0.60	48	65	77	85	550	1970	2990	4340	340	1210	2860	4560
					92.3	1330	0.59	51	69	82	88	680	2340	2830	4290	410	1260	2870	4320
					95.5	1540	0.59	55	74	85	89	620	2240	3360	....	410	1460	3580	....
					81.1	980	0.60	81	95	101	106	2490	4600	4500	5620	2190	3900	4910	6340
L-13-2	57	25	8	7	89.1	1210	0.59	91	102	106	108	3350	5010	5290	5710	2840	4430	5230	6900
					93.3	1370	0.56	99	110	110	111	4220	5720	5460	6240	3310	4600	5240	7040
					95.3	1550	0.55	100	111	111	111	4980	6280	6490	7190	3600	4900	6240	7570
					77.9	920	0.58	50	66	77	80	2050	3930	4820	6750	960	1980	3980	5870
L-20	31	49	3	15	87.1	1170	0.56	56	73	82	84	1900	4010	5130	7180	1360	2610	5020	7030
					92.7	1400	0.35	57	74	83	84	2100	4770	5730	6600	1580	3220	5650	7230
					95.3	1530	0.53	62	76	84	85	2770	5480	6590	7390	1790	3560	5880	7620

[a]Concrete water-cement ratio required to produce 140 per cent flow (24-in. flow table); cement-aggregate ratio 1:5.6 by weight, using 0 to ¾-in. Boulder Dam aggregate.
[b]Mass curing for 28 days, then curing at 70° F. until test.

Referring to Table 3, it is seen that after the age of 3 months the only increase in contribution to heat of hydration is that of C_2S, which apparently continues hydrating up to the age of at least 1 year. However, the contribution of C_2S is still only 60 per cent of that of C_3S. At all ages up to 1 year, the contribution of C_4AF is negligible.

The effect of fineness of cement upon the heat of hydration is shown in Table 4 by the values for 3 commercial cements (30M, 39E, and 15M) and 3 laboratory cements (L-4-2, L-13-2, and L-20), each ground to several degrees of fineness. While the effect of fineness is not the same for all chemical compositions, the average effect is of interest. For the 6 cements investigated, it appears that on the average an increase in the specific surface of 100 sq. cm. per gram (corresponding roughly to an increase of 2.3 in the percentage passing the 200-mesh sieve) increases the heat of hydration by the amounts shown in the last column of Table 3. It is evident that while the effect at the age of 7 days is appreciable, at the age of 1 year it is too small to be of practical significance.

COMPRESSIVE STRENGTH OF MORTAR

The effect of chemical composition upon strength is illustrated by the values of Table 2. The calculated contributions of the four major compounds in cement to the compressive strength of mortar, for laboratory cements of equal fineness, are presented in Table 5. Since in some cases there are large variations between observed values of strength and values computed by the use of these contribution factors, the factors should be considered only as indication of trends rather than as working constants.

TABLE 5—EFFECT OF CHEMICAL COMPOSITION AND FINENESS OF CEMENT UPON MORTAR COMPRESSIVE STRENGTH

Curing Condition	Age	Contribution of Each Per Cent of Compound to Compressive Strength, p.s.i.[a]				Effect of Increase in Specific Surface of 100 Sq. Cm. per G., p.s.i.[b]
		C_3S	C_2S	C_3A	C_4AF	
70° F.	7 Da.	+46	— 1	+14	—11	+220
70° F.	28 Da.	+76	+21	+12	—57	+240
70° F.	3 Mo.	+87	+46	—13	—30	+320
70° F.	1 Yr.	+95	+74	—44	—25	+310
Mass	7 Da.	+73	+ 9	+ 3	—53	+260
Mass	28 Da.	+81	+33	—14	—28	+270
Mass[c]	3 Mo.	+77	+47	—25	— 5	+320
Mass[c]	1 Yr.	+98	+74	—92	—28	+220

[a]For laboratory cements of specific surface 1200 sq. cm. per gram.
[b]Within range of specific surface 1000 to 1600 sq. cm. per gram.
[c]Mass curing for 28 days, then curing at 70° F. until test.

Referring to Table 5, it is seen that at the age of 1 year C_3S is still the major strength-producing compound, and that between the ages of 3 months and 1 year its contribution to strength has increased 9 per cent for 70° F. curing and 27 per cent for mass-concrete curing. During the same period, there is no increase in the contribution of C_3S to heat of hydration (see Table 3). The contribution of C_2S to strength also increases relatively more than the contribution to heat of hydration (60 per cent increase in strength factor vs. 33 per cent increase in heat factor). At the age of 1 year, the probable reduction in strength for an increase of 1 per cent in the C_3A content is very large. As at the earlier ages, C_4AF produces a moderate reduction in strength.

The effect of fineness of cement upon the compressive strength of mortar is shown by the values in Table 4, and the average increase in compressive strength resulting from an increase in specific surface of 100 sq. cm. per gram is shown by the factors in the last column of Table 5. It is seen that the advantage of fine grinding in increasing the strength is maintained at the age of 1 year, the average increase in strength at this age being 265 p.s.i. for an increase in specific surface

of 100 sq. cm. per gram, whereas the corresponding increase at the age of 7 days is 240 p.s.i.

STRENGTH-HEAT RATIO

The strength-heat ratio, expressed as the ratio of compressive strength (p.s.i.) to the heat of hydration (calories per gram of cement) is in one sense a measure of the efficiency of a cement for mass-concrete construction. The approximate effect of chemical composition of the cement upon the strength-heat ratio is shown by the contribution factors of Table 6. A study of this table shows the influence of C_3S to be practically constant at all ages. The beneficial effect of C_2S in producing strength combined with low heat generation is constantly on the increase up to the age of 1 year and at this age it is greater than that of C_3S. The undesirable effect of C_3A upon the strength-heat ratio becomes more pronounced with age.

To show more clearly the effect of composition of cement upon the strength-heat ratio, reference may be made to cement L-20 (see Table 2), a cement of moderately high C_2S and low C_3A content (in general the type of cement now in use in the construction of Boulder Dam). At the age of 1 year this cement has a mortar strength under mass-concrete curing conditions of 7180 p.s.i. and a heat of hydration of 84 cal. per gram, giving a strength-heat ratio of 85 p.s.i. per cal. per gram. Considering cement L-5, a cement of normal C_3S and C_2S content but high in C_3A, it will be noted that at the age of 1 year it has a corresponding strength of 4520 p.s.i. and a heat of hydration of 123 cal. per gram, giving a strength-heat ratio of only 37 p.s.i. per cal. per gram.

The average increase in the strength-heat ratio resulting from an increase in specific surface of 100 sq. cm. per gram is shown in the last column of Table 6. It will be observed that the beneficial influence of fine grinding upon the strength-heat ratio is maintained even at the later ages.

TABLE 6—EFFECT OF CHEMICAL COMPOSITION AND FINENESS OF CEMENT UPON STRENGTH-HEAT RATIO

Age	Contribution of Each Per Cent of Compound to Strength-Heat Ratio, p.s.i. per Cal. per Gram[a]				Effect of Increase in Specific Surface of 100 Sq. Cm. per Gram, p.s.i. per Cal. per Gram[b]
	C_3S	C_2S	C_3A	C_4AF	
7 Da.	+0.8	+0.2	—0.4	—0.2	+2.3
28 Da.	+0.7	+0.6	—0.7	+0.1	+2.0
3 Mo.	+0.7	+0.7	—0.8	+0.4	+2.8
1 Yr.	+0.8	+0.9	—1.4	+0.4	+1.8

[a]For laboratory cements of specific surface 1200 sq. cm. per gram.
[b]Within range of specific surface 1000 to 1600 sq. cm. per gram.
NOTE: Strength specimens mass-concrete cured for 28 days, then at 70° F.

VOLUME CHANGES OF MORTARS

To determine changes in length due to causes other than temperature variation, periodical measurements have been made of unstressed 1½ by 1½ by 12-in. mortar bars containing Boulder Dam sand. The cement-aggregate ratio was 1:3¼ by weight, and the water-cement ratio was approximately 0.58 by weight. The bars were cured under mass-concrete conditions for 28 days, and thereafter were stored either under water at 70° F. or in air of 50 per cent relative humidity at 70° F. Initial readings were taken at the age of 2 days, and the reported values are referred directly to the 2-day lengths.

TABLE 7—EFFECT OF CHEMICAL COMPOSITION OF CEMENT AND FINENESS OF GRINDING UPON VOLUME CHANGES OF MORTAR

Cement No.	Compound Composition, Per Cent				Change in Length of Mass-Cured Mortar Bars, Millionths per Unit		
	C_3S	C_2S	C_3A	C_4AF	28-Day Expansion, Mass Curing	Total Expansion at 1 Yr., Storage under Water at 70° F. after 28 Days Mass Curing	Net Contraction at 1 Yr., Storage in Air of 50% R. H. at 70° F. after 28 Days Mass Curing
L-2	68	9	12	9	136	256	375
L-1	55	23	11	8	71	189	399
L-3	26	52	11	8	102	202	449
L-4	12	67	10	9	88	177	901
L-5	51	25	18	3	65	182	516
L-6	55	24	0	18	65	110	530
L-7	12	70	0	15	88	210	892
L-20	31	49	3	15	105	165	558
Average of 6 coarse cements	Avg. spec. surf.: 1000 cm²/g.				85	157	604
Average of 6 med. fine cements	Avg. spec. surf.: 1550 cm²/g.				80	192	583

Results of the tests are given in Table 7 for selected laboratory cements of specific surface 1200 sq. cm. per gram, and also for the average of 6 coarse and 6 medium-fine cements. It is seen that volume changes are apparently affected by chemical composition of cement and by fineness of grinding. Based upon the more complete data, the following observations appear to be justified:

1. The higher the C_3A content or the higher the MgO content, the greater the expansion under moist conditions.

2. The higher the C_2S content, the less the expansion under water but the greater the contraction in air.

3. The finer the cement, the greater the expansion under moist conditions.

4. The amount of contraction in air is not appreciably affected by variations in fineness.

It should be emphasized that these observations refer only to conditions of mass curing and only to cements within the normal range of fineness.

The most probable contribution to volume changes in mortar of each of the four major compounds in cement has been computed by the method of least squares for the age of 1 year. The values thus obtained are given in Table 8, being based upon clinker analysis of 12 laboratory cements of equal fineness, equal SO_3 content, equal MgO content, and C_2S content less than 65 per cent. Since the probable error of the calculated constants is large, they should be considered only as indications of the trends to be expected, rather than as working constants.

TABLE 8—CONTRIBUTION OF MAJOR COMPOUNDS IN CEMENT TO VOLUME CHANGES OF MORTAR—LABORATORY CEMENTS OF SPECIFIC SURFACE 1200 SQ. CM. PER GRAM

Volume Changes	Contribution of Each Per Cent of Compound to Net Change in Length between Ages of 2 Days and 1 Year, Millionths per Unit of Length			
	C_3S	C_2S	C_3A	C_4AF
Expansion, mass-concrete curing for 28 days, then storage under water at 70° F.	+1.61	+1.12	+9.44	—0.22
Contraction, mass-concrete curing for 28 days, then storage in air of 50 per cent rel. hum. at 70° F.	+4.66	+7.24	—1.36	+3.65

It appears that the greatest influence upon expansion is exerted by C_3A, and that the greatest influence upon contraction is exerted by C_2S.

To determine the effect of prolonging the moist curing period upon the contraction during storage under dry conditions, 9 of the laboratory cements varying widely in composition were transferred from water storage to dry air at the age of 1 year. Comparing the contractions of these bars after 2 months drying with those of similar bars stored in dry air for 2 months after only 1 month of mass-concrete curing, the following results were noted:

1. The *total* contraction for the cements of high C_2S content was greater than for those of normal or high C_3S content, but the difference as between types of cement was appreciably less for the long than for the short curing period.

2. The *net* contraction for the cements of high C_2S content was appreciably less for the long than for the short curing period, although the length of curing period appears to have had little effect upon the contraction of cements of normal or high C_3S content.

DURABILITY OF MORTARS

In an attempt to determine the effect of chemical composition and fineness of grinding of cement upon the resistance of mortars to the action of weather, accelerated tests have been made in which the specimens (2 by 4-in. mortar cylinders) were subjected to alternate high and low temperatures and alternate wetting and drying followed by a test for compressive strength. Prior to the beginning of the durability test, the specimens were cured for 16 months, either under mass-concrete conditions (in sealed containers) or moist at 70° F. The treatment to simulate the action of weather consisted of 60 successive cycles of water-soaking at 70° F., freezing at 0° F., water-soaking at 70° F., and drying in an air draft at 160° F. Specimens were then tested in compression, and the ratio of the strength of treated specimens to that of corresponding untreated specimens has been taken as an indication of the relative durability. The validity of the test as a true measure of the resistance of corresponding concretes to the action of weather over a long period of course cannot be demonstrated, consequently the results should not be accepted too literally.

The results for selected cements are given in Table 9 and the contributions of each principal compound in cement to the durability ratio (the ratio of the strength of treated specimens to the strength of untreated specimens) of 18 laboratory cements are given in Table 10.

TABLE 9—DURABILITY RATIO UNDER CONDITIONS OF ALTERNATE FREEZING AND THAWING AND ALTERNATE WETTING AND DRYING. AGE, 18 MONTHS

Cement No.	Compound Composition, Per Cent				Durability as Indicated by Compressive Strength after Treatment, p.s.i.					
					Mass Curing[a]			Curing at 70° F.		
	C_3S	C_2S	C_3A	C_4AF	Untreated	Treated	Ratio	Untreated	Treated	Ratio
L-2	68	9	12	9	5110	6020	1.18	5900	5620	0.95
L-1	55	23	11	8	....	....	....	5550	5390	0.97
L-3	26	52	11	8	5510	4730	0.86	5870	4930	0.84
L-4	12	67	10	9	4370	3740	0.86	4800	3220	0.67
L-5	51	25	18	3	4110	4230	1.03	4320	3930	0.91
L-6	55	24	0	18	6960	7130	1.02	6570	5550	0.84
L-7	12	70	0	15	5580	5070	0.91	5980	5000	0.84
L-13	55	27	9	6	5840	6590	1.13	6170	7260	1.18
Average for 43 cements					4920	4440	0.90	5260	4350	0.83

[a]Mass curing for 28 days, then curing at 70° F. in sealed containers until treatment or test.

The application of the contribution factors of Table 10 in some cases results in rather large discrepancies between the observed and computed values, and for this reason the factors should be considered only as indications of trends and not as working constants.

TABLE 10—EFFECT OF CHEMICAL COMPOSITION OF CEMENT AND CURING CONDITION UPON DURABILITY RATIO UNDER CONDITIONS OF ALTERNATE FREEZING AND THAWING AND ALTERNATE WETTING AND DRYING. AGE 18 MONTHS

Curing Condition	Contribution of Each Per Cent of Compound to Durability Ratio of Treated to Untreated Compressive Strengths			
	C_3S	C_2S	C_3A	C_4AF
70° F. moist	+0.015	+0.012	—0.005	—0.011
Mass-concrete curing for 28 days, then 70° F.	+0.013	+0.010	+0.007	+0.002

From a study of the values of Table 10 it appears (1) that the chief contributors to a high durability ratio are C_3S and C_2S, pointing to the advantage of cements low in both C_3A and C_4AF, (2) that mass-concrete curing conditions are more favorable to producing a high durability ratio than are standard moist curing conditions, and (3) that other things being equal the durability ratio is somewhat less for cements high in C_4AF than for cements high in C_3A.

Resistance of Standard Mortar to Action of Sodium Sulphate

To determine the resistance of the cements under investigation to the action of sulphate waters, the ends of broken standard (Ottawa sand) briquets from the 28-day tension tests were immersed in 2 and 10-per-cent solutions of sodium sulphate. All of the specimens in the 10-per-cent solution showed the effect of the salt within four months, and all of the specimens in the 2-per-cent solution exhibited signs of disintegration within 8 months.

Considering the effect of chemical composition of cement, it appears that the chief cause of rapid disintegration is C_3A, and that C_2S is more resistant than is C_3S. The greatest resistance is exhibited by cements of high C_2S and low C_3A content.

CONCLUSIONS

The following conclusions are based upon tests of 48 portland cements of 16 commercial brands, some of which were commercially ground to various degrees of fineness and some of which were laboratory ground to a fixed fineness, and 45 laboratory portland cements covering an extreme range of chemical composition and fineness. All of these cements were manufactured under conditions in all respects similar to those under which normal portland cements are produced.

Heat of Hydration

1. The chemical composition of cement has an important effect upon both the rate and total amount of the heat of hydration during any period.

2. Tricalcium aluminate (C_3A) liberates more heat per unit of weight than does any other major compound, and this liberation occurs very largely during the early stage of the hardening process.

3. Tricalcium silicate (C_3S) is next in importance from the standpoint of heat liberated during the hydration process. Its heat is liberated principally during what may be called the "secondary" stage of hardening, which under normal conditions will be within the first week or even within the first day after hardening begins.

4. Dicalcium silicate (C_2S) is third in importance with respect to heat of hydration. The evidence is that the heat of hydration generated by this compound is slow and long-continued.

5. Tetracalcium alumino-ferrite (C_4AF) has very little effect upon the heat of hydration at any age.

6. The effect of an increase in the loss on ignition is a decrease in the heat of hydration. Approximately, this amounts to a decrease at the age of 1 year of 3 calories per gram of cement for each increase of 1 per cent in loss on ignition.

7. Other things being equal, the finer the cement the greater the heat of hydration, but within the normal fineness range the effect is small and becomes less with time. For an average portland cement, an increase in specific surface of 100 sq. cm. per gram, under conditions of mass-concrete curing, will increase the 28-day heat of hydration approximately 2 calories per gram and the 1-year value approximately 1 calorie per gram.

8. Increasing the water-cement ratio increases the heat of hydration, but under average conditions the percentage of increase in heat of hydration is only about one quarter of the percentage of increase in water-cement ratio. The lower the water-cement ratio, the greater the effect of a given change in water-cement ratio upon heat of hydration.

9. For the same water-cement ratio and the *same curing temperatures*, the heat of hydration of a given quantity of cement is practically independent of the richness of mix, being essentially the same in a neat-cement paste as in a corresponding concrete.

Strength-Heat Ratio

10. Considering the ratio of strength expressed in p.s.i. to the heat of hydration expressed in calories per gram (which ratio is in one sense a measure of the efficiency of a cement), other things being equal, regardless of age, the higher the fineness the higher the strength-heat ratio; the lower the tricalcium aluminate content the higher the

strength-heat ratio; and for the later ages, the higher the dicalcium silicate content the higher the strength-heat ratio.

Strength

11. The strength of concrete under both normal and mass-concrete curing conditions is affected greatly by the chemical composition of the cement, but the influence is greatest at the early ages.

12. Tricalcium silicate (C_3S) is principally responsible for the early strength of cement mortars and concretes. A large percentage of its contribution to the strength occurs within the first week of the hardening period.

13. Dicalcium silicate (C_2S) contributes very little to the early strength of cement mortars and concretes, but is responsible for most of the gain in strength after the first week. It provides a long-continued increase in strength so long as water is available for hydration.

14. For those cements which are high in tricalcium silicate, there is but a moderate gain in strength after 28 days; but for cements which are high in dicalcium silicate, there is a large gain in strength after 28 days.

15. Weight for weight, dicalcium silicate contributes about three quarters as much to the strength at later ages as does tricalcium silicate.

16. Other things remaining equal, the greater the tricalcium aluminate (C_3A) content and the less the tetracalcium alumino-ferrite (C_4AF) content, the greater the strength at the early ages but the less the strength at the later ages.

17. The effect of an increase in fineness is to increase the strength, not only at the early ages but also at the later ages. Under mass-concrete curing conditions, the percentage of increase in strength is approximately one half of the percentage of increase in specific surface. Under the condition of moist curing at 70° F., the percentage of increase in strength is approximately equal to the percentage of increase in specific surface. The effect of increase in fineness upon strength is in contrast to the effect upon heat of hydration, which is relatively small.

18. Continuous mass curing results in higher strength at early ages than does curing at normal temperature.

19. For cements which are high in dicalcium silicate, the 7 and 28-day compressive strengths of mortars cured under normal conditions at 70° F. do not give a true indication of the strength of these mortars when cured under mass-concrete conditions. On the other hand, it has been determined that a curing condition of 1 day at 70° F.

followed by a temperature of 100° F. until time of test produces results not greatly different from those which will be obtained under mass-concrete conditions, especially for cements which are high in C_2S content.

Volume Changes

20. The volume changes of mortars cured under mass-concrete conditions are considerably affected by the chemical composition of the cement.

21. In general, other things being equal, the higher the tricalcium aluminate (C_3A) content and the lower the tetracalcium alumino-ferrite (C_4AF) content, the greater the expansion under continuous moist conditions and the less the contraction under conditions of dry air storage.

22. The higher the dicalcium silicate (C_2S) and the lower the tri-calcium silicate (C_3S) content, the greater the contraction in dry air but the less the expansion under water.

23. The finer the cement, the greater the expansion under moist conditions, but within the limits of the tests, variations in fineness do not affect appreciably the amount of contraction in dry air.

Resistance to Weather

24. Under the conditions of test, the "durability ratio" (as defined by the ratio of compressive strength of mortars subjected to alternations of high and low temperatures and to alternations of wetting and drying, to the compressive strength of corresponding mortars not so treated) appears to be considerably affected by the chemical composition of the cement.

25. The major compounds which contribute to a high durability ratio are C_3S and C_2S. Hence, in general, the higher the combined percentage of C_3S and C_2S, and the lower the combined percentage of C_3A and C_4AF, the greater the durability ratio. There is evidence that weight for weight, the contribution to the durability ratio is greater for C_3S than for C_2S, and is greater for C_3A than for C_4AF.

26. Mass-concrete curing conditions appear to be more favorable than 70° F. curing in producing a high durability ratio.

Resistance to Sodium Sulphate

27. The resistance of mortar to the action of sodium sulphate is affected greatly by the chemical composition of the cement. It appears that tricalcium aluminate (C_3A) is the chief cause of rapid disintegration and that dicalcium silicate (C_2S) is more resistant than is tri-calcium silicate (C_3S). Hence the greatest resistance to the action of

sodium sulphate is exhibited by cements of high C_2S and low C_3A content.

Water Requirement

28. Other things being equal, the higher the C_3A content the greater the water requirement to produce a given consistency of concrete, although this increase in water requirement is not large. It is not apparent that variations in the amounts of the other major compounds appreciably affect the water requirement for concrete of given consistency.

29. Within the usual range of fineness of commercial cements, the finer the cement the less the water requirement to produce a concrete of fixed consistency.

30. There is no consistent relationship between the water required for normal consistency of neat-cement pastes and the water required to produce a given consistency of corresponding concretes.

For such discussion of this paper as may develop, readers are referred to the JOURNAL *for November-December 1934 (Proceedings Vol. 31). Discussions should be available to the Secretary by October 1, 1934.*

Design of Two-Way Slabs on Beams

*Report of Committee 302**

BY E. H. UHLER,† AUTHOR-CHAIRMAN

The theoretical formula by H. M. Westergaard and the proposed emperical formulas of the Chicago, New York, Boston, and German Building Codes are at variance in the design of two-way slabs on beams, whereas the tests of Slater and McMillan show in most cases, that both the approximate theoretical formulas of Westergaard and the building codes are safe. The number of tests made do not give enough points so that curves can be constructed.

APPROXIMATE THEORETICAL FORMULA—WESTERGAARD THEORY

For details of moments and stresses in slabs by Professor Westergaard and the late W. A. Slater see *Proceedings* American Concrete Institute, Vol. 17, 1921, or National Research Council Bulletin No. 32.

The approximate Westergaard formulas for bending moments per unit width are given in Table 1.

*This report has been submitted to critic members of the Committee of whom the following have approved: F. E. Brown, S. G. Cutler, Inge Lyse, May, 1934.

†Assistant Professor of Civil Engineering, Lehigh University, Bethlehem, Pa.

TABLE 1

Bending moments of rectangular homogeneous plates either supported, fixed, or supported and fixed on four sides, as determined by H. M. Westergaard.

Bending Momemts Per Unit Width of Rectangular Plates Supported on Four Sides.

Case	Moments in Short Span		Moments in Long Span	
	Neg. Mom. at Centre of Edge	Pos. Mom. at Centre of Slab	Neg. Mom. at Centre of Edge	Pos. Mom. at Centre of Slab
1 Four Edges Simply Supported	0	$\frac{1}{8(1+2m^3)} wS^2$	0	$\frac{m^2(1+m^2)}{48} wL^2$
2 Short Span Fixed Long Span Simple	$\frac{1}{12\ (1+0.2m^4)} wS^2$	$\frac{1}{24\ (1+0.4m^4)} wS^2$	0	$\frac{(1+0.3m^2)m^2}{80} wL^2$
3 Long Span Fixed Short Span Simple	0	$\frac{1}{8(1+0.8m^2+6m^4)} wS^2$	$\frac{m^2}{8(1+0.8m^4)} wL^2$	$\frac{0.015m^2(1+3m^2)}{1+m^4} wL^2$
4 All Spans Fixed	$\frac{1}{12(1+m^4)} wS^2$	$\frac{1}{8(3+4m^4)} wS^2$	$\frac{m^2}{24} wL^2$	$m^2 \overline{0.009\,(1+2m^2-m^4)}\, wL^2$

$m = \frac{\text{short span}}{\text{long span}} = \frac{S}{L}$ w = dead load + live load per square foot

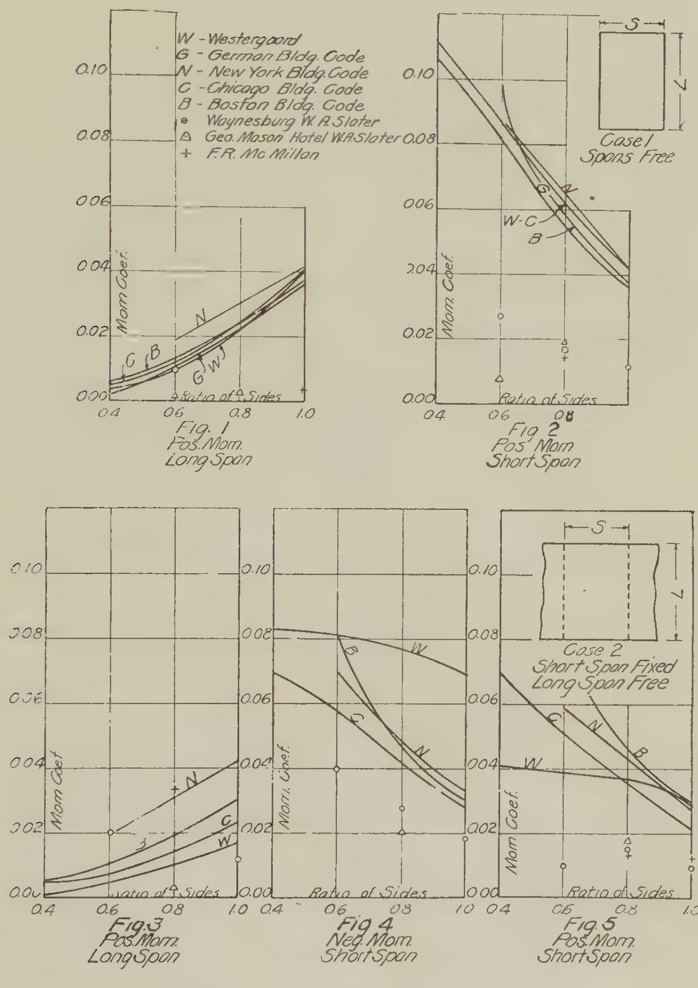

FIG. 1-5

In Fig. 1 to 12 the bending moment coefficients for the various codes and formulas have been plotted as ordinates, and the ratios of the short span to the long span as abscissas. The curve of coefficients representing the Westergaard theory is designated by the letter "W." The building codes usually limit the ratios of the sides. Therefore only curves of ratios from 0.4 to 1.0 have been constructed.

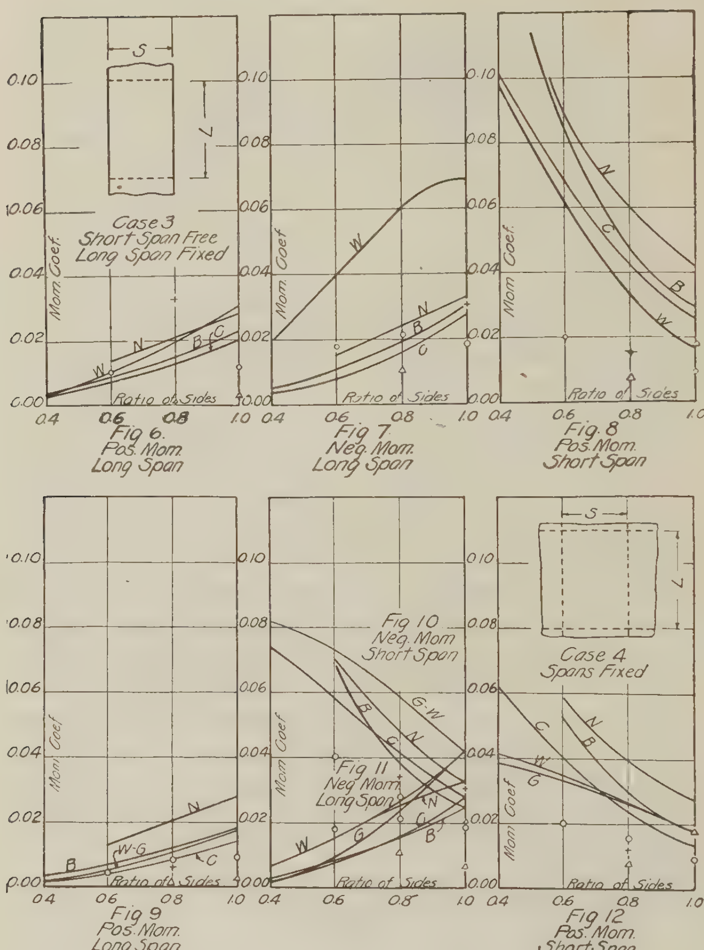

FIG. 6-12

BUILDING CODE FORMULAS

The bending moments for floor slabs supported on beams as proposed for the Chicago, New York, German, and Boston building codes are shown in Tables 2, 3, 5, 6, respectively. Numerical values of coefficients for various ratios of short span to long span have been computed and graphically shown in Fig. 1 to 12.

TABLE 2—PROPOSED CHICAGO BUILDING CODE

Bending Moment Per Unit Width of Two-Way Reinforced Concrete Floor Slabs Supported on Four Sides.

Case	Moments in Short Span		Moments in Long Span	
	Neg. Mom. at Centre of Edge	Pos. Mom. at Centre of Slab	Neg. Mom. at Centre of Edge	Pos. Mom. at Centre of Slab
1 Four Edges Simply Supported	0	$\frac{1}{8(1+2m^3)} wS^2$	0	$m^2 \frac{(1+m)}{48} wL^2$
2 Short Span Fixed Long Span Simple	$\frac{1}{12(1+2m^3)} wS^2$	$\frac{1}{12(1+3m^3)} wS^2$	0	$m^2 \frac{(1+m)}{80} wL^2$
3 Long Span Fixed Short Span Simple	0	$\frac{1}{8(1+4m^3)} wS^2$	$m^2 \frac{(1+m)}{72} wL^2$	$m^2 \frac{(1+m)}{96} wL^2$
4 All Spans Fixed	$\frac{1}{12(1+2m^3)} wS^2$	$\frac{1}{12(1+5m^3)} wS^2$	$m^2 \frac{(1+m)}{72} wL^2$	$m^2 \frac{(1+m)}{144} wL^2$

$m = \frac{\text{short span}}{\text{long span}} = \frac{S}{L}$ $\qquad w$ = dead load + live load per square foot.

TABLE 3—PROPOSED BUILDING CODE FOR NEW YORK CITY

Bending Moments Per Unit Width of Two-Way Reinforced Concrete Floor Slabs Supported on Four Sides.

Case	Moments in Short Span		Moments in Long Span	
	Neg. Mom. at Centre of Edge	Pos. Mom. at Centre of Slab	Neg. Mom. at Centre of Edge	Pos. Mom. at Centre of Slab
1 Four Edges Simply	0	$\frac{e_s r_s}{8} wS^2$	0	$\frac{e_L r_L}{8} wL^2$
2 Short Span Fixed Long Span Simple	$\frac{2e_s r_s}{20} wS^2$	$\frac{e_s r_s}{12} wS^2$	0	$\frac{e_L r_L}{8} wL^2$
3 Long Span Fixed Short Span Simple	0	$\frac{e_s r_s}{8} wS^2$	$\frac{2e_L r_L}{20} wL^2$	$\frac{e_L r_L}{12} wL^2$
4 All Spans Fixed	$\frac{2e_s r_s}{20} wS^2$	$\frac{e_s r_s}{12} wS^2$	$\frac{2e_L r_L}{20} wL^2$	$\frac{e_L r_L}{12} wL^2$

$e_L = e_s = e = \frac{2}{4\text{-}k}$ $\qquad k = \frac{\text{long span}}{\text{short span}} = \frac{L}{S}$ $\qquad r_L = \frac{1}{1 + \left(\frac{CL}{FS}\right)^3}$ $\qquad r_s = 1 - r_L$

$$C = F = \begin{cases} 1.00 \text{ for simple spans} \\ 0.87 \text{ for semi-continuous spans} \\ 0.76 \text{ for fully-continuous spans} \end{cases}$$

w = dead load + live load per square foot.

The values of $e_s r_s$ and $e_L r_L$ for three ratios of k are computed in

The coefficient curves are designated by the first letter of the name which they represent; that is, Chicago by the letter "C," German by the letter "G," New York by the letter "N," Boston by the letter "B."

TABLE 4

k	$e_s = e_L = e = \frac{2}{4-k}$	$\frac{CL}{FS}$	$\left(\frac{CL}{FS}\right)$	$r_L = \frac{1}{1+\left(\frac{CL}{FS}\right)^3}$	$r_s = 1 - r_L$	$r_s e_s$	$r_L e_L$
1.67	0.857	$\frac{1}{1} \times \frac{15}{9} = 1.67$	4.67	0.177	0.823	0.705	0.152
1.25	0.728	$\frac{1}{1} \times \frac{15}{12} = 1.25$	1.95	0.339	0.661	0.483	0.246
1.00	0.666	$\frac{1}{1} \times \frac{15}{15} = 1.00$	1.00	0.500	0.500	0.333	0.333

TABLE 5—GERMAN BUILDING CODE

Bending Moments Per Unit Width of Two-Way Reinforced Concrete Floor Slabs Supported on Four Sides.

Case	Moments in Short Span		Moments in Long Span	
	Neg. Mom. at Centre of Edge	Pos. Mom. at Centre of Slab	Neg. Mom. at Centre of Edge	Pos. Mom. at Centre of Slab
1 Four Edges Simply Supported	0	$\frac{V_1}{8} w_s S^2$	0	$\frac{V_1}{8} w_L L^2$
2 Short Span Fixed Long Span Simple				
3 Long Span Fixed Short Span Simple				
4 All Spans Fixed	$\frac{1}{12} w_s W^2$	$\frac{V_4}{24} w_s S^2$	$\frac{1}{12} w_L L^2$	$\frac{V_4}{24} w_L L^2$

w = dead load + live load per square foot

Load in Short Direction $W_s = w \frac{L^4}{S^4 + L^4}$ $\qquad V_1 = 1 - \frac{5}{6}\left(\frac{S^2L^2}{S^4 + L^4}\right)$

Load in Long Direction $w_L = w \frac{S^2}{S^4 + L^4}$ $\qquad V_4 = 1 - \frac{5}{18}\left(\frac{S^2L^2}{S^4 + L^4}\right)$

TESTS ON SLABS SUPPORTED ON FOUR EDGES

The results of the floor tests in the George Mason Hotel at Alexandria, Va., made by the late Professor Slater are shown in Fig. 1 to 12 by a triangle (Δ). The details of these tests may be found in the Institute of Research Circular No. 41, a Lehigh University publication.*

Results of floor tests also made by Professor Slater for J. J. Whitacre of Waynesburg, Ohio, and given in detail in the Technologic Paper No. 220 of the Bureau of Standards, Washington, D. C., are shown by means of a sign (⊙) in Fig. 1 to 12.

The results of tests made by F. R. McMillan of the Dennison Two-Way Reinforced Concrete Floor System at the Dunwoody Building, Minneapolis in 1924, are shown in Fig. 1 to 12 by a plus sign (+).

*See, *Proceedings* American Concrete Institute, 1930 Vol. 26 , page 286.

TABLE 6—PROPOSED BOSTON BUILDING CODE

Bending Moments Per Unit Width of Two-Way Reinforced Concrete Floor Slabs, Supported on Four Sides.

Case		Moments in Short Span		Moments in Long Span	
		Neg. Mom. at Centre of Edge	Pos. Mom. at Centre of Slab	Neg. Mom. at Centre of Edge	Pos. Mom. at Centre of Slab
1	Four Edges Simply Supported	0	$\frac{3}{10}(1.0)\ \frac{1}{8}\ \frac{1}{m^2}\ wS^2$	0	$\frac{3}{10}(1.0)\ \frac{1}{8}\ m^2\ wL^2$
2	Short Span Fixed Long Span Simple	$\frac{3}{10}(1.2)\ \frac{1}{12}\ \frac{1}{m^2}\ wS^2$	$\frac{3}{10}(1.2)\ \frac{1}{16}\ \frac{1}{m^2}\ wS^2$	0	$\frac{3}{10}(0.8)\ \frac{1}{8}\ m^2\ wL^2$
3	Long Span Fixed Short Span Simple	0	$\frac{3}{10}(0.8)\ \frac{1}{8}\ \frac{1}{m^2}\ wS^2$	$\frac{3}{10}(1.2)\ \frac{1}{12} m^2\ wL^2$	$\frac{3}{10}(1.2)\ \frac{1}{16}\ m^2\ wL^2$
4	All Spans Fixed	$\frac{3}{10}(1.0)\ \frac{1}{12}\ \frac{1}{m^2}\ wS^2$	$\frac{3}{10}(1.0\ \frac{1}{16}\ \frac{1}{m^2}\ wS^2$	$\frac{3}{10}(1.0)\ \frac{1}{12} m^2\ wL^2$	$\frac{3}{10}(1.0)\ \frac{1}{16}\ m^2\ wL^2$

$$m = \frac{\text{short span}}{\text{long span}'} = \frac{S}{L}$$

Under the direction of the supervising architect, tests on a number of post office and hospital buildings have been made for the purpose of satisfying the specifications for deflections. Bending moment coefficients were not determined..

CONCLUSION

The information gathered from various sources and superimposed on the same diagrams, Fig. 1 to 12, justified the statement made at the beginning of this report; that formulas and tests are at variance in the design of two-way slabs on beams.

Upon examination of Fig. 1 to 12, and due regard to the limited experimental data available, Professor Westergaard's approximate theoretical curves are apparently the basis of all the various building codes studied, except the German Code which incidentally agrees well with Westergaard's analysis. Therefore, since Professor Westergaard's curves have a rational background, the writer recommends their use without modifications in building codes.

For such discussion of this report as may develop, readers are referred to the JOURNAL *for November-December 1934 (Proceedings Vol. 31). Discussions should be available to the Secretary by October 1, 1934.*

Two-Way Slabs in the Proposed Building Code for Boston and New England*

BY JOHN R. NICHOLS†
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In setting out to select from a rather large field a method of design for two-way slabs, or to devise a method, the Sub-Committee on Reinforced Concrete held in mind a number of qualities which such a method must possess to merit adoption.

First, it must be simple. We considered wholly unsuitable for use in a code any method which involved long or complicated formulas or the use of charts or tables unless of the simplest possible kind. We did not underrate the valuable work of Westergaard and others in analysing the stresses in two-way slabs or in correlating analysis with the results of tests, but we demanded a simpler expression of these results than that which he had developed, approximate or empirical if need be, for use in the proposed code. The methods chosen for Chicago and New York and that used by the Bureau of Yards and Docks, of the Navy, seemed to us unnecessarily cumbersome.

Second, it must be related to the design of one-way slabs. The two-way slab is after all a slab subject to bending and there appeared to the committee no reason for not dealing with it as such in accordance with well established methods. Adherence to familiar forms fosters that sense of proportion which helps the designer to avoid errors. The process of blindly putting physical data into a mechanical formula and turning the crank was abhorrent to all of us.

Third, it must accord substantially with the facts as revealed by analysis and test, to yield safe construction consistently and with reasonable economy. Our knowledge of the physical data involved is not such as to justify laborious attempts at theoretical accuracy with loss of simplicity and clarity of process.

Fourth, it should avoid obvious irrationality if this can be done without loss of more important characteristics.

*Received by the Secretary, Oct. 6, 1933.
†Boston, Mass.

After some study of a number of methods of design, we noted that the relative bending in the two directions was affected by three factors.

1. Restraint or continuity at the ends of the span is a factor affecting bending with which we have long been familiar. Our committee had already made provision for dealing with it in connection with one-way slabs. For this reason, and because it helps the designer to visualize his problem at all its stages, we decided to evaluate the bending in terms of $w\ L^2$ and to introduce a coefficient for each of the three factors that affect the result. The coefficient covering restraint, which we called C_1, would be that already determined for one-way slabs. By the same means we took care of the negative bending at supports.

2. The shape of the panel in plan, whether square or rectangular, as determined by the ratio of its sides, has a marked effect on the relative bending in the two directions. Most of the methods of design studied by the Committee used the symbol r to represent the ratio of the long to the short span, its value, therefore, being greater than one. In all methods the bending in the short direction was given in terms of r by one expression and the bending in the long direction by another. The two expressions were not always complementary, in fact in some cases they were obviously unrelated so that the curves for bending on a given span, as the lateral dimension of the panel varied from less to greater than the span, suddenly changed direction and form. This seemed to us needlessly irrational, and we decided to avoid any method that implied other than a smooth variance from one limit to the other of the useful range of the ratio of the panel sides. These considerations and the desire again to adhere to familiar forms led us to seek an expression for bending in either direction in terms of the span in that direction, whether long or short. We let r represent the ratio of the lateral dimension or width of the panel to the span in question, a value which may be greater or less than one.

3. Relative restraint on the sides of a panel affects the bending. Thus, if a square panel is equally restrained on all four sides the bending is the same in both directions. But if two opposite sides are free while the other two are restrained, the slab in the restrained or continuous span will be more rigid and will consequently assume a larger responsibility in carrying load. We have recognized this by introducing the factor, C_o, which adds 20 per cent to the responsibility of the span continuous at both ends and subtracts 20 per cent from that of the span continuous at neither support. The percentage is 10 when one span has continuity at one support, the other at neither or both.

The most formidable problem which the Committee faced was to establish a relation between bending and the shape of the panel. In considering all the existing methods and a host of formulas suggested by members of the Committee and others, we were early attracted by the simplicity of one of the methods for determination of the bending in the long span, namely a constant in terms of the short span. Translated into the symbols adopted for the code, this formula for a single panel freely supported on all sides becomes:

$$M = \frac{1}{24} r^2 w L^2 = \frac{1}{3} r^2 \frac{w L^2}{8}$$

which had been used only for values of r less than one. Extending this curve to values of r greater than one, we find it lies reasonably close to values established by other commonly accepted methods when r is low, but tends to high values for M when r rises above 1.3. To compensate for this it was suggested that the whole curve be lowered 10 per cent and with this modification the formula was adopted. Clothed in the form which we consider it should have, it is presented in the proposed code, as follows:

$$M = 0.30\, C_o\, C_1\, r^2 w L^2$$

After this expression was proposed in its final form, it was given careful scrutiny by the Committee with the following general conclusions:

The expression is manifestly empirical. It does not pretend to be otherwise. It adheres to a familiar form, however, and introduces separate coefficients to deal with the several factors affecting the bending: C_1 to deal with restraint or continuity in the span, positive or negative bending; C_o to deal with continuity at the lateral edges of the panel and $0.3\, r^2$ to deal with the shape of the panel.

The bending given by the formula is generally within reasonable range of the results of other accepted methods; in some cases greater, in others less. Where the variation is considerable the Committee is of the opinion that the results of its method are closer to the facts than those of other methods. The bending so established is larger by a substantial margin than that to be found by Mr. Westergaard's analysis, and is generally larger than that already allowed in Boston for certain types of cellular two-way slab construction. If, indeed, the bending runs high for values of r in excess of 1.5, the Committee is content so to discourage two-way construction in long, narrow panels. It is not necessary to place an arbitrary limitation upon the length of the panel in terms of its width, for the bending on the short span is the same as for one-way slabs when r reaches 1.82.

Worthy of attention also is the method of determining the load on the supporting beams of the panel and correspondingly the shear in the slab. This also is a function of the shape of the panel and the relative restraint at the sides and ends of the span. Using the same symbols the expression for the load supported by a strip of slab one unit wide spanning in a given direction is

$$W = \tfrac{1}{2} C_o \, r \, w \, L$$

For a panel with equal restraint on four sides C_o is one and the load supported on a square unit of area by a strip spanning in the short direction varies from half the unit load in a square panel to the whole unit load when the width is twice the span. The load on a square unit area supported by a strip spanning in the long direction varies from one half the unit load in a square panel to one quarter the unit load in a panel whose width is one half the span. The coefficient, C_o takes care of the added responsibility assumed by the slab which has a greater degree of continuity in the direction of its span than laterally.

The most attractive feature of the method is its simplicity which is rather remarkable. This may not at first appear to those accustomed to some other method, for the familiar always appears simple. If curves or tables are used, all methods are equally simple, once the tables are computed, the curves drawn. It is in the absence of such devices that the simplicity of a method is seen in its right proportions. If we observe that $r\,L$ in the formula is the lateral dimension of the panel at right angles to the span, we may use this value directly and it is not necessary to compute r at all. C_o is then chosen from five possible values and the rest is the same as for a slab of one-way construction. Those few engineers who make frequent use of two-way slab construction will in any event set up tables or curves in the forms which seem most attractive to them. It is not for such that we sought simplicity. Those engineers whose practice deals constantly with one-way slabs and occasionally with two-way slabs will find the simplicity we have achieved, especially the adherence to the usage of one-way construction, most helpful. It may well be that this will lead to wider use of two-way slab construction in circumstances when economy is so served.

The provisions of the proposed code are as follows:

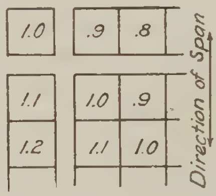
Concrete slabs, either solid, ribbed, or combination slabs, supported on four sides by beams, girders or walls, and reinforced to span in two directions, shall be designed in accordance with the following provisions.

(a) The slab shall be regarded as consisting of a series of adjacent strips of unit width spanning in each direction. In computations for shear and diagonal tension, bond, and for the loading of supporting members each strip, spanning in either the

longer or the shorter direction, shall be assumed to carry and transmit to its supports a total load W, represented by the expression:

$$W = \tfrac{1}{2} C_o\, r\, w\, L = \tfrac{1}{2} C_o\, w\, L_1,$$

in which w is the load per unit area of the slab
L is the span of the strip
L_1 is the width of the panel transverse to L
r is the ratio of L_1 to L
C_o is a coefficient dependent on the position of the panel relative to adjacent panels continuous with it at its ends and sides, as indicated on the following diagram of panels. (Fig. 1) Full restraint at an end support, as defined in Section 2642, shall be considered equivalent to continuity in determining C_o.



(b) The positive bending moment for a strip of unit width in the middle half of the panel, spanning in either direction, shall be assumed as given by the following expression:

$$M = 3/10\, C_1\, C_o\, r^2\, w\, L^2 = 3/10\, C_1\, C_o\, w\, L_1^2$$

in which C_1 is a coefficient for bending determined in accordance with the conditions of restraint at end supports of the strip as provided in Sec. 2642.*

*Section 2642, besides defining moderate and full restraint and the conditions under which arbitrary coefficients for bending may be used, contains the following table:

TABLE OF ARBITRARY COEFFICIENTS FOR BENDING

Conditions of Restraint	No. of Spans	End Span: End Sup.	End Span: Mid-Span	First Int. Sup.	Interior Spans: Mid-Span	Interior Spans: Other Int. Supports
Case 1. Slabs or beams with negligible restraint at end supports	1	1/24	1/8			
	2	1/24	1/10	1/8		
	Mult.	1/24	1/10	1/9*	1/16	1/12
Case 2. Slabs or beams with moderate restraint at end supports	1	1/16	1/10			
	2	1/16	1/12	1/9		
	Mult.	1/16	1/12	1/10	1/16	1/12
Case 3. Slabs or beams with full restraint at end supports	1	1/12	1/12			
	2	1/12	1/16	1/10		
	Mult.	1/12	1/16	1/11	1/16	1/12

*1/10 for slabs.

(c) Negative moments at and adjacent to supports between two panels shall be determined by the same expression given in (b) for positive moment; taking C_1 as the coefficient for negative moment determined according to Sec. 2646, and in case the conditions of continuity in the two panels are different, taking the mean value of C_o.

(d) The bending in strips of unit width in the outer quarters of the panel may be assumed to be one-half that of the strips of the middle half.

(e) Lines of inflection in a two-way panel shall be assumed at a distance of one-fourth of its shorter span from supports over which the slab is continuous.

For such discussion of this paper as may develop, readers are referred to the JOURNAL *for November-December 1934 (Proceedings Vol. 31). Discussions should be available to the Secretary by October 1, 1934.*

Publication of Discussion Deferred

Publication of discussion, originally intended for this issue, is now scheduled for the September-October issue (*Proceedings* Vol. 31) under the following titles:

Stresses at a Crack, Size of the Crack and the Bending of Reinforced Concrete

Simplified Concrete Mix Design

Plastic Flow in Plain and Reinforced Concrete Arches

Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's corps of Reviewers.

Originally published in the *News Letter* pages of the JOURNAL (Sept.-Oct., 1933 to May-June, 1934), these reviews are here issued to members as a supplement, to form a part of Vol. 30, *Proceedings*, American Concrete Institute—following the last *Proceedings* page of the JOURNAL for May-June 1934. Beginning with JOURNAL Vol. 6, 1934-35, *Current Reviews* will be published in each JOURNAL issue as a part of the *Proceedings* pages—EDITOR.

"Aggregate grading and concrete quality"

By H. N. WALSH, Institution of Civil Engineers of Ireland, *Proc.*, 1933, pp. 275-338.

Reviewed by J. C. PEARSON

PERSONS who are interested in the classic problem of ideal grading of aggregates, as well as in the practical problem of making good concrete, will be interested in this paper by Professor Walsh (M. E., University College, Cork, Mem. A. C. I.).

Author states that the most important properties of concrete are: (1) Workability when freshly mixed, (2) Density when placed, (3) Suitable cement content, (4) Strength when cured. Strength is put last because the best modern cements are so good that concrete made with them may be more than strong enough for its purpose and yet be porous or even honeycombed. Strength is no longer a sufficient criterion of concrete quality, hence it is now more necessary to devote attention to aggregate gradings than formerly. Where strength is a secondary consideration, and density, impermeability and economy are of primary importance, grading of aggregate becomes the controlling factor.

Author's work was based on a study of gradings for maximum density, or slightly less than maximum density accompanied by workability, as determined by Fuller, Bolomey, Dutron and others. He was not so much concerned with maximum density, however, as with limiting gradings for the production of good quality concrete in rich, medium and lean mixes. His idea was to fix 3 zones of grading, a central zone such that any grading in it would produce good concrete with a medium

cement content; below this a zone of coarse gradings for rich mixes of high strength; and above it a zone containing such proportions of fine particles as to give sufficient workability in lean mixes. Zone boundaries were determined by trial, involving tests of many artificially graded mixtures. Establishment of these zones enabled the author to prepare tables showing the minimum cement to mixed aggregate to give easily workable concrete with "type-gradings," that is, with gradings having the general trend of the zone boundaries.

It is not easy to describe the author's type-gradings without diagrams, but those who are familiar with the Furnas-Anderegg papers (*Ind. & Eng. Chem.* 23, 1058-64, Sept. 1931—not referred to by Professor Walsh) will note a similarity between the curves of those writers for ideal grading and the type-gradings of this paper—cumulative curves of increasing slope when plotted against log spacing of aggregate sizes.

Some of the deductions from the author's studies are particularly interesting. For example, his diagrams show that the ¾ in. maximum aggregate requires more fine aggregate with a given cement content than the ½ in. maximum, in the same manner that optimum gradings for lean mixes require more fines than for average or rich mixes. They show that the commonly specified proportions $1:n:2n$ by loose volume will not give good concrete with ¾ in. maximum aggregate when n is greater than 1½; but the allowable value of n will be higher when 1½ in. maximum aggregate is used. They show also that limiting the portion of the fine aggregate to a certain small percentage passing the No. 50 sieve is illogical, as this fraction should vary both with the richness of mix and the grading of the coarse aggregate.

Strength of concrete was not a major item in this investigation, but the author shows how his diagrams may be used for design of mixes, basing strength on the cement-water ratio by weight. His results for designed strength (with rather poor curing conditions in respect of temperature control) fell within 15 per cent of a straight line represented by an equation of the type $S = K\ (C/W + K')$.

"The mobility of concrete and the economical use of cement"

LIONAL NORMAN JAMIESON, B. C. E. *Journal, Institution of Engineers Australia.* July, 1932 V. 4, No. 7, p. 225.

A VALUABLE and interesting approach to the problem of designing concrete mixtures from the standpoint of workability and yield. The work of placing is said to be in three parts: 1. Overcoming internal segregation, 2. Working out entrained air, and 3. Rectifying surface segregation. The work in performing 2 or 3 usually suffices for 1, and the total work is therefore 2 or 3 whichever is the greater. Where 2 is greater than 3 the grading may be altered to give greater yield without increasing the labor of placing and greater economy will obtain. The lack of a proper standard of workability is noted and a measure is improvised which is simply the number of prods of a rod necessary to place concrete properly. For *plasticity* of mix the author uses R_3, the ratio of the absolute volume of all of the materials excepting the coarse aggregate to the voids in the coarse aggregate. Low R_3 means low plasticity. Cost problems involving labor of placing and cement content are illustrated. A term "Unit water demand" is introduced which may be defined as the water demand per unit absolute volume of a solid constituent for a given slump. It may be expressed for cement alone or for mortar or concrete mixtures. It depends upon size, shape of particle, grading, and special surface properties of the solid material.

Test results are given which show parabolic relations when the unit water demand is plotted as ordinates and the volumetric water cement ratio minus 0.425 is plotted as abscissae. For 28-48 sand;

$$(y\text{-}0.10)_2 = 0.116x$$

A theoretical equation of mobility is worked out mathematically and checked experimentally for the total water required for a unit volume of concrete. It is made up of a term for the cement, a term for the aggregate covering action sometimes spoken of as fluxing, and a term for the frictional restraint requirements or lubrication. With this formula a simple calculation is all that is necessary after the water cement ratio is determined upon. Considerable more experimental work is necessary to determine the constants involved.

"Tests of anchorage for reinforcing bars"

By CHESLEY J. POSEY. University of Iowa Studies in Engineering, Aug. 15, 1933. 31 pages.

Reviewed by ARTHUR R. LORD

THIS bulletin describes pull-out tests of bars embedded in 2000-lb. concrete. The slip at the *beginning* of the anchorage was measured "on the run," i. e., without stopping the testing machine. The embedded ends were either straight (plain, deformed, threaded or indented) or bent into circular hooks (radius equal to 10, 11, 12, 13½, 16, 20 or 28 bar diameters) and were 22 bar diameters long (11 in. for ½-in. round bar, 13¾ in. for ⅝-in. round bar). Two, four or seven rows of indentations were made by hand, using a blunt cold chisel, on the embedded end of straight bars.

Ratios of load to slip are reported for various ratios of stress in the bar to ultimate crushing strength of concrete in the block. Hooked ends of the length tested were found to provide insufficient anchorage to develop yield point strength of the bars. Hooks of smaller radius slipped more at low loads but built up an *increasing* resistance to slipping at high loads, while hooks of larger radius slipped less at low loads but showed a *decreasing* resistance to slipping after a slip of from 0.03 to 0.04 in. had occurred.

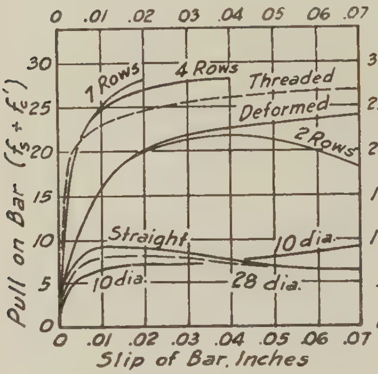
Commercial deformed bars and bars with two opposite rows of closely spaced indentations showed similar resistances to slipping at low loads, while the deformed bars were superior at higher loads. Bars with straight ends, *threaded*, provided adequate anchorage with much less slipping, but are not practical for use in design. Bars with either 4 or 7 rows of indentations were greatly superior to the deformed or threaded bars and the test embedment of 22 bar diameters proved sufficient with these bars to develop fully the yield point strength of the bars, at slips of 0.01 in. or less. It is believed that this new type of deformed bar is suitable for commercial production and will simplify anchorage details greatly. The number and spacing of the indentations should be a function (not yet fully determined) of the shearing strength of the concrete engaged between adjacent indentations and the bearing strength of the concrete on an indented surface. Failure to balance these two resistances to slipping are believed to explain the lower anchorage value of the commercial deformed bars now available.

The accompanying sketch illustrates the relationships between the various types of end anchorages in a general way. It has been prepared by the reviewer and does not appear in the original bulletin. In the sketch the symbols are:

10*d*—Hook with radius of 10 bar diameters, plain bar.

28*d*—Hook with radius of 28 bar diameters, plain bar.

Str—Straight end, plain bar.
Def—Commercial deformed bar, manufacturer not named.
2 Rows, 4 Rows, 7 Rows — Number of rows of closely spaced indentations for full length of embedded end of bar.
f_c'—Ultimate compressive strength (about 2000 p. s. i.).
f_s —Computed stress on bar at beginning of anchorage.



Unusual structures in Europe

Reviewed by A. A. BRIELMAIER

SOME recent structures put up by continental engineers are rather unique or outstanding, and might be of interest to JOURNAL readers. The following are a few instances which show that progress is being made along conventional lines as well as in new directions.

One of the chief difficulties contended with in airplane hangar design is the necessity for unobstructed space. The new hangar at Lyon-Bron (France) (Gaston Le Marec, *Le Genie Civil*, Vol. 52, No. 11, Sept. 10, 1932) is a distinct achievement in meeting this requirement. Overall dimensions are 164 by 131 ft. The longer sides are closed by walls while the shorter are provided with sliding doors. The only supports other than the end walls are two columns, 33-ft. apart, midway between them. The columns are part of a two-story (37 ft. high) frame, connected by two trusses with each wall. These four trusses, in turn, support at 22-ft. intervals the roof trusses cantilevering out 65 ft. from the column faces. The clear height under the bottom chord of the cantilevers ranges from 27 ft. at the building line to 17 ft. at the column. A roof slab with the hardly excessive thickness of 2 in. follows the curve of the lower chord of the roof cantilevers. Only one unit has so far been erected, but the design permits the construction or addition of any multiple of units.

A commercial arcade in Munich (Craemer, *Beton U. Eisen*, Vol. 32, Heft. 3, Feb. 5, 1933) is covered by unusual barrel roof arches (13-ft. span, 4-ft. rise, 2.5 in. thick) are of concrete with glass inserts 4 in. in diameter and 2.5 in. deep, spaced permit placing a 60 deg. three-way reinforcing system; one 7mm round in the main direction and one 5mm round in each of the diagonals. From their lowly beginning as a part of sidewalk lights it seems that glass units are joining the modern movement towards higher structures.

An arch bridge over the Euphrates (A. Wilhelm, *Beton U. Eisen*, Vol. 32, Heft. 7-8, April 5, 1933) has been erected by a Swedish-Danish company for the Turkish Government. The arch (354-ft. span and 84-ft. rise) has the ends fixed in the rock shore cliffs. Columns carry the load from the roadway (21 ft. wide) to the arch barrel which varies in depth from 4.5 to 7.5 ft. and in width from 15.7 to 19.7 ft. In section the barrel consists of a box frame with an intermediate vertical wall, all reinforced. The thickness of the members is about one foot. The hollow section is stiffened under the columns with a membrane wall. A suspension system, hung from high towers over the abutments, provided

the means of transporting the materials and also supported the arch centering.

The problem of saving weak retaining walls and abutments from threatened failure has received the attention of a Berlin engineer who presents a clever solution (A. Schroeter, *Beton U. Eisen*, Vol. 32, Heft. 10, May 20, 1933). In principle it is merely the substitution of concrete slabs for the relieving arches which are not unknown in this connection. The backfill is removed to the depth indicated by the condition of the wall and a concrete slab, preferably of precast units, is supported at the wall and at some distance from it on the newly formed backfill slope. At intervals the slab should be anchored to the wall, but in a hinged or loose manner so that a settlement of the earth-supported end will not have serious results. The statical effect of the construction is to increase the vertical load on the wall and to remove the lateral earth pressure for a distance below the relieving slab, dropping the resultant of the earth pressure and reducing the overturning moment. By means of various arrangements and numbers of these relieving slabs, the earth pressure may be prevented, either entirely or to any desired extent from acting on the wall.

Since the war there has been considerable activity in Italy in the construction of continuous girder, reinforced concrete bridges over the wide rivers of the plains regions (G. Escher, *Beton U. Eisen*, Vol. 32, Heft. 10, May 20, 1933). Among the larger of these are the two over the Crati river, one with seven spans of 66 ft. and one with 12 spans of 67 ft.; the Coscile river structure with six spans of 55 ft.; the Roganello river bridge with seven spans of 56 ft. The continuity is broken at every third support by expansion joints. The appearance of the structure is that of series of flat arches, due to the curved soffits of the girders whose depth at the piers is between two and three times that at mid-span.

Current Reviews

Report on prevention of dusting of concrete floor surfaces

By Concrete Sectional Committee, adopted by Science Committee, issued by Council of the Institution of Structural Engineers, London, England, July, 1933, 8 pages.

A summary of the recommendations:

(1) All aggregate must be tough, well graded, clean and free from dust.

(2) Proportions of materials: 1 part cement, 1½ parts of fine aggregate, 3 parts coarse aggregate, using the class of aggregate recommended.

(3) High-grade portland or aluminous cements are suitable.

(4) Use clean water and avoid using too much, 5 gallons per cwt. of cement when the recommended mix is used.

(5) Do not trowel or float more than absolutely necessary.

(6) Polishing, if required, should be done after the paving is hard.

(7) Do not sprinkle dry cement on the surface.

(8) Keep the concrete thoroughly wet for at least five days.

Effect of curing conditions on the strength of cement mortar

D. O. Woolf and K. F. Shippey. *Public Roads*, Aug., 1933, Vol. 14, No. 6, pp. 106-108. Reviewed by F. H. Jackson.

Tension and compression tests made on Ottawa sand mortar prepared with 3 normal portland cements and 1 high early strength portland cement with ages of test from 7 days to 1 year. Specimens stored in running water, still water, and moist air. Tension specimens stored in water were subject to retrogression in strength after 28 to 180 days, while those stored in moist air showed steady gain in strength. Compression specimens of normal portland cement (2 inch cubes) seldom showed retrogression to age of 1 year, but those made with H. E. S. cement were affected after 90 days storage in water. Retrogression definitely attributed to solution of part of the cement by storage water. Tests do not show any reason for changing from present standard practice of curing mortar specimens in water to age of 28 days. Moist air storage recommended for tests at greater ages.

Mission Inn, Riverside, California

American Architect, Sept., 1933, pp. 15-24. Reviewed by Rexford Newcomb

Views of new additions to the Mission Inn, Riverside, California, by G. Stanley Wilson, Architect, covering a city block and practically all of reinforced concrete, was left as it came from the forms. The material made it possible to develop ornament as integral parts of the structure and offered a practical method of incorporating the many grills, columns, windows and bits of old statuary that formed an important part of the owners' collection.

French specifications for hydraulic cements

Revue des Materiaux de Construction et de Travaux Publics, Aug., 1933. Reviewed by P. H. Bates.

The new tentative specification of the French Standardization Society (Association Francaise de Normalisation) which will be a matter for discussion and possible adoption this fall as a new French standard, is noteworthy in that it places under one set of standards and one set of testing methods nine different groups of hydraulic cements, comprising 36 different grades. The following will indicate very briefly what these different groups of cements are and the compressive strengths (numerals which appear after

each grade) which they must develop to meet grade requirements. Where 3 sets of numerals are given, the first indicates the strength at 48 hours; the second at 7 days; and the third at 28 days. Where 2 sets appear, they represent the strength at 7 and 28 days. These values are expressed in kilograms per square centimeter and may be converted to pounds per square inch by multiplying by 14.5. The specimen used is a 5 cm. cube made of a 1:3 by weight grade of sand-mortar, the consistency of which appears to be somewhat wetter than our standard consistency.

Group 1—Hydraulic Limes (Chaux hydrauliques)—Hydraulic limes result from the burning of more or less argillaceous limestone and reducing the product to a powder through slaking or slaking and grinding with or without the addition of ground grappiers (grappiers are those parts of burned argillaceous limestone which resist slaking).

Class A—Hydraulic lines, as defined above.

Grade 1—Hydraulic	5.	12.5
Grade 2—Strongly hydraulic	12.5	25.
Grade 3—Strongly hydraulic for special purposes	31.5	63.

Class B—Hydraulic lime—slag blends (Chaux au laitier). This class is composed of those hydraulic limes to which have been added at least 30 per cent of ground quenched basic blast furnace slag.

Grade 1—Standard	12.5	25.
Grade 2—Special	31.5	63.

Group 2—Slag Cements (ciments de laitier)—The slag cements result from prepared mixtures, perfectly homogeneous and finely ground, of granulated quenched basic blast furnace slag with either—

Class A—Not more than 30 per cent of slaked or hydraulic lime, or

Class B—Not more than 15 per cent of portland cement.

There are two grades in both classes:

Grade 1—Standard	50	100
Grade 2—Special	100	160

Group 3—Natural Cements (ciments naturels)—Natural cements result from the grinding of burned natural mixtures of argillaceous limestones of uniform composition.

Class A—Rapid setting.

Grade 1—Ordinary	12.5	20.	31.5
Grade 2—Standard	20.	31.5	50.
Grade 3—Special	31.5	50.	80.

Class B—Slow setting.

Grade 1—Ordinary	12.5	25.
Grade 2—Standard	25.	50.
Grade 3—Special	50.	100.

Group 4—Grappier Cements (ciments de grappiers)—Grappier cements are produced by the grinding of grappiers resulting in the manufacture of well burned limes after hydrating the lime and removing the grappiers from it.

Class A—Grappier cements.

Grade 1—Ordinary	12.5	25.
Grade 2—Standard	25.	50.
Grade 3—Special	50.	100.

Group 5—Portland Cements (ciments artificiels)—Portland cements are obtained from mixtures composed principally of carbonated lime, silica, alumina, and oxide of iron carefully proportioned, chemically and physically homogeneous, burned almost to fusion, and then ground to a powder. (Agreement has not been reached on the amount of additions subsequent to burning for regulating the set and other properties. There is agreement that the maximum SO_3 should not exceed 3 per cent. There is the possibility of there being allowed other additions than those added as sulphates.)

Class A—Portland cement.

Grade 1—Standard	100.	160.
Grade 2—Special	160.	250.

Class B—High early strength cements.

Grade 1—Ordinary	100.	250.	315.
Grade 2—Special	160.	315.	400.

Group 6—Blended Cements (ciments composes)—Blended cements are produced from carefully controlled homogeneous mixtures, finely ground, of either any of the several varieties of hydraulic cements or of hydraulic cements and chemically inert materials, or natural minerals.

Class A—Mixed cements (ciments mixtes). Mixed cements are mixtures of grappiers and natural or portland cements.

Class B—Diluted cements (cements amaigris) are mixtures of portland cement and inert minerals. There are 3 grades in each of these classes.

Grade 1—Ordinary	12.5	25.
Grade 2—Standard	25.	50.
Grade 3—Special	50.	100.

Class C—Pozzuolanic cements (ciments aux pouzzolanes) are mixtures of hydraulic cement and pozzuolana other than blast furnace slag.

Grade 1—Standard	80.	160.

Group 7 — Portland Cement-Slag Blends (ciments metallurgiques)—The portland-slag cements are produced by mixing and fine-grinding carefully prepared mixtures of portland cement and basic granulated water-quenched blast furnace slag.

Class A—High portland-low slag mixtures (ciments de fer) are mixtures of portland cement and slag in which the per cent of slag is between 15 and 30.

Class B—Low portland-high slag mixtures (ciments de haut fourneau) are mixtures of slag and portland cement in which the per cent of slag is between 30 and 85. There are two grades in each of these classes of cement.

Grade 1—Standard	100.	160.
Grade 2—Special	160.	250.

Group 8—High Sulphate Slag Cements (ciments metallurgiques sursulfates) are produced by adding to basic granulated water-quenched blast furnace slag small quantities of portland cement or slaked lime and sulphates in such quantities that the per cent of SO_3 shall exceed 5. There are two grades in this group.

Grade 1—Standard	100.	160.	
Grade 2—High early strength	100.	250.	315.

Group 9—High Alumina Cements (ciments alumineux) are produced by grinding, after heating to or almost to fusion, mixtures composed principally of alumina, silica, oxide of iron, and lime or carbonate of lime. They should contain at least 30 per cent oxide of alumina. There is but one grade in this group of cements.

Grade 1	315.	355.	400.

Durable concrete structures

Bulletin No. 14, Austrian Concrete Committee (1933). Reviewed by INGE LYSE.

A SUBCOMMITTEE report on durability of concrete structures includes studies on water requirements, water-cement ratio, density and cement content of concrete mixes and also discusses gradation of aggregates and workability of concrete. Some of the more important results of the studies are summed up in the following items.

1. The gauging of the correct water content is the outstanding requirement for durable structures, both for proper placibility and for the compressive strength and other qualities of the concrete.
2. A knowledge of the gradation is necessary for estimating the correct water requirement.
3. General recommendations cannot be given for the gradation and the proportion of mixes.
4. It is often advantageous to use jumps in the gradation curves.
5. Powers' remolding apparatus gives satisfactory workability indications.
6. Abrams' water-cement ratio law contains, in spite of its simplicity, all important factors for the quality of the concrete.
7. Accurate proportioning by weight is essential for the production of a durable concrete.

The 52-page report contains a number of tables, diagrams and photographs.

Analysis of unsymmetrical concrete arches

CHAS. S. WHITNEY, *Proc., Am. Soc. C. E.*, Vol. 59, No. 8, Part 1, p. 1247 (Oct., 1933). Reviewed by H. J. GILKEY.

THIS IS an extension of a former paper by the same author* and is itself in the nature of a summary, the full investigation being on file in the Engineering Societies Library, New York.

The problem of the unsymmetrical arch usually arises in connection with the end spans of bridges where it may be desirable to vary the elevation better to meet the grades at the approaches. The method of analysis is of general application and the treatment is divided into three parts. The first to determine the elastic properties

*Design of Symmetrical Concrete Arches. Transactions Amer. Soc. C. E., Vol. 88 (1925) p. 931.

of the rib segments each side of the crown; second, the determinations of the reactions of the complete rib and third, a numerical example accompanied by a short discussion of unsymmetrical arch design. The paper restricts itself to what the author terms "the geometry of the problem" and does not include the selection of values of Young's modulus, movement of abutments or the selection of coefficients for thermal expansion, shrinkage, plastic flow, etc. Formulas are given for the determination of their effects upon the reactions, however.

It is of special interest to members of the Institute to note that the author of this paper was the recipient of the Institute's Wason Medal, for his 1932 convention paper, "Plain and Reinforced Concrete Arches" as well as the recipient of the Am. Soc. C. E. J. James R. Croes Medal in 1925 for his earlier paper on the design of symmetrical concrete arches. It is reasonable to expect therefore that those who are interested in problems relating to the arch will wish to study this paper.

Russia builds rail-bridge over Dnieper river

C. L. Christensen, (Consulting Engr. for Mostatrust during construction; formerly designing Engr., Niagara River Arch, Mich. Central Ry.), *Eng.-News Record*, Aug. 10, 1933, p. 160, 161. Reviewed by N. H. Roy.

Present-day Russian methods of design and construction are illustrated in a bridge over the Dnieper River. The bridge consists of 31 reinforced concrete arches, 4 reinforced concrete continuous girders, and 2 steel through-trusses over the navigation channels. All arches consist of two ribs connected by diaphragms. The ribs of the longer arches (170 ft.) are in a vertical plane (no batter) and the lateral forces from trains and wind loads are transferred to the tops of high piers through the floor slab. The ribs of the shorter arches rest on low piers and are given transverse batter to provide against lateral forces of wind and train side sway. Each of the shorter arches has a completely independent superstructure. All arches were poured simultaneously. The falsework trusses for the river spans were built as three-hinged arches, some of timber, some of steel, and all are to be used again on another bridge. Reinforcing bars in the arch ribs were spliced by welding. The finished structure was tested for deflections and vibrations under the load of two heavy freight locomotives on the single track. On the 170-ft. spans the average deflection was about $\frac{3}{32}$ in. and the horizontal movement at the floor-slab expansion joints over the piers was $\frac{1}{8}$ in. Before opening to traffic, both locomotives, coupled together ran over both the tangent and curved sections of the bridge at 50 m. p. h.

Volume changes in concrete

O. Goffin and G. Mussgnug, *Zement*, Vol. 22, No. 40, p. 549-56, 1933. Reviewed by Inge Lyse.

A discussion of the factors which control the volume changes of concrete and a report on experimental studies of the effect of the composition and fineness of the cement in which the most outstanding results are summarized as follows:

The volume changes are principally affected by the amount of free lime, the fineness of the cement and the additions of gypsum and calcium chloride. High free lime content increases the expansion and subsequently reduces the shrinkage. Very fine cements show greater shrinkage than do coarser cements. More than four per cent gypsum admixture increases the expansion and slightly reduces the shrinkage. Calcium chloride increases the shrinkage but has little effect upon the expansion. The silica content has practically no effect upon the volume changes.

Design of joints in concrete pavements

R. D. BRADBURY, Twelfth *Proceedings*, Highway Research Board, 1932, pp. 105-136 (issued Sept. 1933). Reviewed by R. W. CRUM.

THIS paper is a comprehensive discussion from a theoretical standpoint of an item in the design of concrete pavements that is usually handled in an arbitrary manner based upon individual experience rather than upon rational analysis. The author shows that the various elements of the design are susceptible to analysis and supports his conclusions with investigational evidence.

Predetermined slab size is essential to any rational analysis of the stresses occurring in concrete pavements, and each slab must be structurally adequate to resist the external forces and to act with its neighbors in such a way as to maintain the integrity of the whole pavement. The design therefore becomes a problem in crack control within the slabs and provision for adequate joints.

There are two basic types of joints, closed or hinged and open or free. The closed joint is one which resists transverse shear but not bending moments. Its shear resisting feature depends upon some sort of face interlock. To assure this effect tie bars must be used to prevent separation of the slabs. Open joints are free to open or close with the slab movements. They are not resistant to bending moments but may be made shear resistant by means of slip dowels. Random cracks that are likely to occur in any slab can be made to function as closed joints by the use in the design of such reinforcement as will provide closely spaced units distributed throughout the slab area.

A common arrangement is to provide alternate expansion and contraction joints allowing for expansion of at least one inch per 100-ft. of pavement, and joint widths not greater than 1 to 1½ in.

Proper spacing of transverse joints depends upon such factors as aggregates, curing, subgrade, and reinforcement, and is largely a matter of experienced judgment. Current practice ranges from 30 to 60 ft.

There is a surprising lack of agreement among highway engineers with respect to the use and design of joints, and it is evident that present practice is largely based upon arbitrarily chosen details rather than upon structural principles. The paper mentions briefly the more common details used by the State highway departments.

The author then passes on to the design of tongue and groove joints, dummy joints and tie bars for closed joints; and edge strengthening, corner strengthening, and slip dowels for free joints. He states that 8,000 lbs. is the maximum wheel load most generally used in the design of joints and recommends an impact allowance of 50 per cent. Some of the more important formulas developed are as follows:

NOTATION

In developing certain formulas, herewith presented as applicable to the various features of joint design, the following notation is used.

W = maximum wheel load (lbs.).
P = load-transfer capacity of a single dowel (lbs.).
L = free length or width of slab (ft.).
w = average weight of slab (lb. per sq. ft.).
h_e = exterior-edge thickness of slab (ins.).
h_c = interior thickness of slab (ins.).
l = length of tie bar or dowl bar (ins.).
s = spacing of tie bars or dowel bars (ins.).
z = maximum width of joint (ins.).
d = diameter of bars (ins.).
A_s = cross-sectional area of steel (sq. ins. per ft. width).
f_s = tensile or bending stress in steel (p. s. i.).
f'_s = shearing stress in steel (p. s. i.).

S = bending stress in concrete (p. s. i.).
f_c = bearing stress on concrete (p. s. i.).
M = bending moment (in.-lbs.).
c = coefficient of subgrade friction.

Tensile resistance of tie bars — $A_s = \frac{Lwc}{2f_s}$ ....(1)

Total length of tie bar — $\int = 80d$.(2)

Edge thickness — $h = \sqrt{\frac{3w}{S}}$ ..(3)

Required bar diameter for edge strengthening for round bar,

$$d = \sqrt{\frac{1.4\ Sh_e^2}{f_s\ (h_c - 2)}} \ldots\ldots (5)$$

for square bar, $d = \sqrt{\frac{1.1\ Sh_e^2}{f_s\ (h_c - 2)}}$ ..(5a)

Required area of steel for corner strengthening, $A_s - \frac{2.2\ Sh_e^2}{f_s\ (h_e - 2)}$ ....(6)

Length of dowels

$$-\int = d\ \sqrt{\frac{20\ f_s}{f_c}} \ldots\ldots (10a)$$

Dowel spacing for 20-ft. pavements

$$S = \frac{120\ P}{W} \ldots\ldots (11)$$

Formulas for calculating values of P as limited by shear, bending and bearing are also given.

The analysis indicates that the center of a dowel bar should in general be at least three diameters from the nearest slab surface.

In concluding the author discusses principally the function of slip dowels, which is to cause the two edges formed by an open pavement joint to deflect simultaneously, and relates the theoretical considerations concerning them to the available test data. A list of references is appended.

Tests of aggregate interlock at joints and cracks

A. C. Benkelman, formerly Research Engineer, Mich. State Highway Dept., Lansing, *Eng.-News Record*, Aug. 24, 1933, pp. 227-232. Reviewed by N. H. Roy.

The author reports test data obtained by the Michigan State Highway Department in 1931 and 1932 on plain, mesh, and bar mat pavements. The effectiveness in carrying shear or the ability to transfer load from the loaded to the unloaded side of joints and cracks was determined by means of an 18,000 lb. axle load which could be applied to the pavement on either side of a crack and a deflectometer for measuring deflections of the slab on a 20-in. gage length on a line perpendicular to the direction of the crack or joint. The ratio of the deflections of the slab on each side of crack or joint when one side is loaded is considered a measure of the percentage of load transferred. Tests were made on the same joints and cracks in the summer, fall and winter. Uncracked pavement will give equal deflections, or the percentage of load transferred across the crack (of zero width) will be 50. It was found that the interlocking of aggregates in small cracks of plain pavement was very effective in transferring load or carrying shear. Cracks in plain pavement opened more in winter than in summer and consequently carried much less load; as much as 66 per cent less. Cracks in reinforced pavement carried very high percentages of load in all seasons. Regardless of the number of transverse cracks in a slab of reinforced pavement their widths were about the same (from 0.04 to 0.05 in.) but for plain concrete the widths of cracks diminished as the number of cracks in a slab increased; for one crack to a slab, the average width of crack was 0.15 in., for two cracks to a slab the average was 0.10 in., etc. Longitudinal joints, deformed metal and dummy joints,

closed and open, were tested and were found to transfer a high percentage of load. The longitudinal reinforcing appeared to prevent opening of cracks beyond a certain fixed limit. Pavement design is discussed and an economical and efficient slab design illustrated.

Proposed French specification for concrete block

Prepared by Committee on Masonry of the General Committee on Building of the French Standardization Society (AFNOR); *Revue des Materiaux de Construction et de Travaux Publics*, Sept., 1933. Reviewed by P. H. BATES.

THE specification covers the dimensions and physical requirements for both cinder and sand-gravel blocks. Four different types are specified, all 40 cm (16 in.) long by 20 cm (8 in.) high, in any of the four following widths: 5, 10, 15 and 20 cm.

The physical requirements demand that for the sand block the absorption should be less than 16 per cent; for the cinder block less than 20 per cent. The compressive strength in the case of the cinder block, on the average, should be more than 60 kg. per cu. cm. (870 p. s. i.). No block in any lot should give a strength of less than 50 kg. per cu. cm. (725 p. s. i.). The compressive strength of the sand blocks, on an average, should not be less than 80 kg. per cu. cm. (1160 p. s. i.), and no one specimen should give a strength of less than 60 kg. per cu. cm. (870 p. s. i.).

Two other interesting tests are presented—one called a stability test and the other a freezing and thawing requirement. In the stability test no cracking should develop after 3 hours building of the block.. In the wet condition, furthermore, the strength cannot be less than 70 per cent of that shown according to the dry tests.

In the freezing and thawing tests the loss of weight cannot exceed one per cent after the blocks have been frozen and thawed 25 times.

Austrian concrete researches

Bulletin 13, Austrian Concrete Committee (1933). Reviewed by INGE LYSE.

REPORTS on tests of reinforced columns. Professor Saliger reports the results of an investigation on high strength steel reinforcement in columns, and Dr. Emperger presents a study on the deformation limit of concrete and the utilization of the longitudinal reinforcement. Professor Saliger's results are substantially in agreement with the results of the Institute's column investigation. However, the effectiveness of the spiral reinforcement is found to be somewhat higher in Saliger's results than in the Institute tests. The ultimate strength of the columns is given by the formula:

$$P = A_c.f_c' + 1.1 \times A_s f_s + 2.8\, A_s' f_s'$$

The effectiveness ratio of the spiral reinforcement is thus 2.8. Professor Saliger proposes the following formula for the design of reinforced concrete columns:

$$P_p = A_c.f_c + A_s.f + 1.1 \times A_s' f_s'$$

where P_p = premissible working load, f_c and f are permitted working stresses in concrete and longitudinal reinforcement. Dr. Emperger presents the following formula for the strength of reinforced concrete columns:

$$P = A_c.f'_c + A_s.f_s + k\,.A_s' f_s'$$

where k should be taken as 1.5. His proposal for a design formula is:

$$P_p = (A\text{-}A_s)f'_c + A_s.f + k\,.A_s' f$$

It is of interest to note that in both these reports the proposed design formulas are based on the working stresses of the concrete and the steel. They are therefore of a different nature than those proposed by either the majority or the minority of the Institute's Committee on Reinforced Concrete Columns, both of which were based on ultimate strength and arbitrary factors of safety.

Loading tests on a new composite type short span highway bridge combining concrete and timber in flexure

R. H. Baldock and C. B. McCullough. Oregon State Highway Dept., Technical Bulletin No. 1, 160 pages. Reviewed by F. E. Richart.

This bulletin describes an investigation of composite T-beams (timber stems, concrete flanges) to be used in highway bridges of spans less than 30 ft. Such bridges, with the overhanging concrete roadway acting as a waterproof and fireproof covering for the timber stringers and pile bents, furnish a type intermediate in cost between plain timber trestles and reinforced concrete bridges.

The tests were made principally to determine the effectiveness of various types of shear connections necessary to secure integral action of the timber and concrete elements. Five types of shear connections were used: (1) short sections of steel pipe, (2) large spikes, (3) rectangular steel plates, (4) daps in timber stem, and (5) combination of daps and spikes. All metal connections were set in timber at right angles to plane of horizontal shear. The connections found most desirable were the steel pipes and the combined daps and spikes. The ultimate strength of composite beams with well designed connections was about twice that of the timber stems alone. The deflection of the composite beam under a given loading did not exceed one-fourth that of the timber stem alone. The effects of repeated loading and of temperature changes were determined. The tests indicated that stresses within working limits could be analyzed by use of the "transformed section."

Examples of bridge designs are given, together with specifications covering grading and selection of lumber and improved methods of preservative treatment. The bulletin closes with an analysis of costs and a discussion of the type of location and traffic to which this type of bridge is suited.

Effect of size of specimen, size of aggregate and method of loading upon the uniformity of flexural strength tests

W. F. Kellerman. *Public Roads*, Jan., 1933, Vol. 13, No. 11, pp. 177-184. Reviewed by F. H. Jackson.

Reports the results of a series of tests to obtain data for use in standardizing a laboratory method for testing concrete beams in flexure. The need for such a procedure is apparent when it is realized that various agencies which use the beam test for determining the quality of pavement concrete employ different procedures with the result that data from different laboratories can not be correlated. The purpose of these tests was to determine which of the several methods of procedure would produce the most uniform results. Variables investigated were method of loading, size of specimen, span length, maximum size of aggregate. The coarse aggregate was crushed limestone used in four different gradings. A typical concrete paving mix was used. Beam specimens were fabricated by rodding in 3-in. layers, each layer being rodded 60 times per square foot of area. Three sizes of specimens were made, 6 x 6 x 21 in., 6 x 6 x 30 in. and 8 x 8 x 27 in. All specimens were tested as simple beams, the rate of loading being held constant at 150 p. s. i. per minute. Both center and third point methods of loading were employed. In the case of the center loading method the bending moment was calculated at the center of span and, also, at the plane of fracture. The span length was three times the depth, except in the case of the 30-in. beams where it was 4½ times the depth.480 beam specimens were made

and tested in connection with this investigation. The results obtained were used as a basis for the recommendation that the third point method of loading and a cross section of 6 x 6 in. be standardized for laboratory work. No definite recommendation was made regarding span length.

Influence of temperature upon the strength development of concrete

N. DAVEY, Bldg. Research Technical Paper No. 14, Dept. of Scientific and Industrial Research (England), Oct., 1933. Reviewed by F. E. RICHART.

THIS PAPER reports several recent investigations on the effect of temperature, due to both external and internal conditions, upon the strength development of concrete made with modern types of cement. The three principal divisions of the paper describe (1) The effect of heated materials, (2) The effect of curing and maturing temperatures, and (3) Strength development in mass concrete.

The use of cement received hot from the mill apparently had little effect upon the time of set or upon the compressive strength of concrete, even though lots of cement specially obtained for the purpose had temperatures as great as 250° F. The effect of preheating materials to produce concrete at 158° F. was compared with results from normal mixtures mixed at 60°F., using a rapid-hardening portland cement. The high temperature produced a large acceleration in strength during the first three days, but no advantage in strength thereafter. The influence of three months storage of normal and rapid-hardening and high-alumina cements at various temperatures was studied. Samples were placed in air-tight containers and maintained at 158° F., 14° F., and at a normal warehouse - temperature. Concrete made with these cements and tested at various ages showed no serious deterioration, though it is noted that ordinary containers would not long withstand the high temperature employed. In the second investigation, concrete made with normal, rapid-hardening and high-alumina cements were cured at carefully maintained temperatures. Normal cements exhibited a regular increase in strength (at ages of 1, 3, 7 and 28 days) with increase in curing temperature from 35 to 95° F. Rapid hardening cement showed a similar trend, but with a wide spread in 1-day strengths produced in this temperature range. The strengths were markedly low for temperatures below 50° F. The effect of curing temperature was not great at the 28-day age. Similar tests with high-alumina cements showed the typical rapid development of strength, almost reaching the maximum at 7 days, with curing temperatures of 36 to 63° F. With temperatures above 68° F the 3-day strength was decreased and was followed by great retrogression in strength at later ages. Tests with sealed specimens showed the retrogression to be independent of water content of mix. Tests with curing in steam at 212° F showed some acceleration of strength with rapid hardening cements, and the elimination of retrogression with high-alumina cements.

To study the heat evolution and strength variation in mass concrete, a small sample of the concrete was placed under adiabatic conditions and used to control the temperature of the storage bath to produce conditions as they would exist inside a large mass of concrete. In this apparatus, a difference of temperature between control specimen and bath, in which thermocouples were inserted, produced a deflection of a galvanometer mirror. This in turn threw a beam of light along a series of thermo-electric cells and served to switch on an electrical heating

unit in the water bath. The normal rise in temperature of the specimen, with no gain or loss in heat, was thus permitted, and this storage temperature was also maintained for auxiliary test specimens.

Temperature records were obtained under adiabatic conditions for normal, rapid-hardening and high-alumina cement concrete, and cylinder tests were made at ages of 1, 2 and 3 days. The portland cements showed accelerated strengths (as compared to ordinary curing at 62.5° F.) while the high-alumina cements generally showed retrogression in strength after 1 day. The definite increase in strength with temperature rise, and the linear relation between cement content and temperature rise for portland cements, were not found for high alumina cements in mass concrete. Thus while high temperatures may be found at the center of a large mass of concrete, increased strengths will be found in this region with portland cements and decreased strengths with high-alumina cements. These mass concrete tests cover only a 3-day period.

Lining the coast range tunnel for Hetch Hetchy water

Eng.-News Record, July 27, p. 107-110. Reviewed by N. H. Roy.

Circular concrete lining for 10.5 ft. water-supply tunnel was designed to meet variable earth pressures encountered in 28.6 miles of tunnelling through diverse rock formations. Heavy and unstable formations necessitated heavy timbering before lining with concrete for much of the distance. For several miles timbering was not sufficient, so pneumatically placed concrete sublining was resorted to. Very little of the timbering could safely be removed before placing concrete lining; hence it was left in place. The design of the concrete lining inside of timber lining was based upon the estimated ultimate strength of the timber lining. When timbering had successfully withstood a section of ground for a long time without undue distortion its ultimate strength was estimated and this strength was considered to be a fair working stress for the design of a concrete lining. In the case of sections where concrete sublinings were used, careful measurements were made with strain gages. The deflections and movements observed were used as a basis for design of the concrete lining to be placed inside the sublining. Strength calculations were based upon the heavy concrete rings between sets. The range of thicknesses of the lining was from a 6-in. minimum in front of 8 x 8-in. timber to 36-in. at the heaviest section. The design was changed by varying the strength of the material and the strength of the concrete (from 3000 to 4000 lb.). There are several interesting problems in connection with this work such as driving operations, paving of the invert, and placing of sidewalls.

Corewall of 31 caissons sunk under air

Eng.-News Record, Aug. 24, 1933, pp. 215-219. Reviewed by N. H. Roy.

A Corewall 1500 ft. long for an earth fill dam has been constructed by sinking 9 x 45 ft. concrete caissons to depths of 40 to 140 ft. in valley drift composed of sand, gravel, silt, water, and boulders. The caissons are sunk end to end allowing about 18 in. between ends in addition to a recess. These spaces are to be excavated and filled eventually by concrete keys. The caissons are of solid concrete except for two muck shafts, a man shaft, and a working chamber at the bottom. Heavy reinforcing is used around the perimeters of the shafts and working chamber. A steel cutting edge is

provided. The working chambers are to be sealed with concrete and the shafts with impervious earth. A crawler crane with a clamshell bucket makes the initial excavation for the caisson sinking. The caisson is built and sunk in sections, the cutting edge—working chamber sections sinking first. A double section of 32-ft. of concrete is then poured on top of the first section and the caisson sunk. Subsequent lifts are 16 ft. high and the 208 cu. yd. of concrete is placed in about 7 hours. A head-frame on top of the caisson is arranged to hoist muck buckets up through both material shafts. Each caisson is landed on bed rock. Continuous pumping of water from near the bottom of an exploratory caisson has lowered the water level 30 ft. and through two subsequent pumping caissons it has lowered the level 50 additional ft. Consequently the air pressure needed has not to date exceeded 18 lb., the critical pressure for comfort and cost.

The supporting strength of rigid pipe culverts

By M. G. SPANGLER, Iowa Engineering Experiment Station, Iowa State College, Ames, Ia., Bulletin 112. Feb. 8, 1933. 100 pages. AUTHOR'S ABSTRACT.

THE TREND of practice in the design of rigid circular pipe culverts has been toward more definite field load calculation and the utilization of laboratory tests for the determination of the field supporting strength of such structures. These tests, however, do not give directly a measure of the structural strength of the pipe when installed under embankments, since the loading conditions in the tests and in the field installations are radically different. It is necessary, therefore, to know the correlation between the laboratory test strength of pipe and the field strength of similar pipe in order to apply the test strengths to problems in design.

The field supporting strength of rigid pipe is dependent upon the distribution of the applied vertical loads produced by the covering earth and by traffic loads at the surface of the embankment, upon the construction conditions affecting the distribution of the vertical reaction and upon the distribution and magnitude of the active lateral earth pressure on the pipe.

The ratio of the field supporting strength to the three-edge bearing laboratory strength of similar pipe is defined as the load factor.

The purpose of this research has been to determine the load factor for rigid pipe culverts when installed under various field conditions affecting the vertical reaction and the active lateral earth pressure, and subjected to vertical loads due to the covering earth, with or without loads due to surface traffic.

The plan pursued to accomplish this purpose was to conduct a number of experiments in which several pipe sections, selected at random from a given shipment, were tested in the laboratory with the three-edge bearing test. A like number of similar sections were then loaded in the field by an actual embankment and the field strength determined. The ratio of these two strengths is the load factor for that set of pipe for the conditions under which they were installed in the field tests.

With the data thus secured as a basis, a rational theory for determining the load factor under all conditions of loading was developed. Working values of the load factor, determined in accordance with this theory, are proposed for a range of field conditions covering all cases likely to be encountered in practice.

Effect of Southern California earthquake upon buildings of unit masonry construction

Report by Raymond E. Davis, chairman, to members Committee C-12 on Mortars for Unit Masonry of the American Society for Testing Materials.

This, a preliminary report, of impressions "on the ground," examining more than 150 buildings after the earthquake of March 10, 1933, will be supplemented by a complete report based on data from laboratory examinations of samples of material. The preliminary report finds "the greatest single factor contributing to this damage was without doubt the very inferior quality of the masonry mortar." These are typical excerpts from the report:

". . . mortars appeared to be deficient in cementing materials. They were porous and were so fragile that they could be crumbled with the fingers. In many instances where a wall had fallen, it was impossible to find any two bricks adhering to one another, and the individual bricks with adhering mortar could be brushed clean with the hand."

"Where frequent headers were provided and failures of this type occurred, it was evidently because of poor workmanship with joints between face and backing not flushed full, or due to poor mortar. It appears that the ordinary metal tie is totally inadequate as a means of anchorage; also, it appears that galvanized iron ties may soon rust out where the mortar is porous. It was observed that where the two portions of such a wall were well bonded together with frequent headers, and where the mortar was of good quality with joints flushed full, this type of construction was satisfactory."

"In the case of some of the poorly constructed buildings, with long unsupported walls, all that remained was a pile of loose brick and broken timbers, there being virtually no mortar bond and the dislodged joint material resembling sand rather than mortar."

"Examination of the materials indicated that the masonry units were of good quality, but that the mortar was almost totally lacking in bond strength. Of the portions that had fallen, it was impossible to find two bricks adhering to one another.

The severe damage to this building may be attributed first to the inferior quality of mortar, and second to lack of unity and rigidity of design."

Bituminous cements

Walter Dyckerhoff, *Das Betonwerk*, Vol. 21, No. 40, 41, Oct. 1 and 8, 1933. Reviewed by Inge Lyse.

These cements are produced by the addition of small amounts of bituminous material, from two to six per cent, to the ordinary portland cements. Special compressed air methods are used for the thorough mixing of the two materials. The resulting product has nearly the same fineness as the original portland cement and microscopic examinations revealed a high degree of uniform distribution of the bituminous material in the cements. These bituminous cements required a higher water content for normal consistency and longer time for initial and final set than did the corresponding portland cements. The test results showed that the compressive, tensile and flexural strengths were less for the bituminous cements than for the portland cement and also gave lower modulus of elasticity. However, they showed a considerably lower shrinkage, especially at early ages, than did the portland cements. Consequently these cements become important in our search for a cement with low volume changes.

Ohms of resistance measure concrete curing

SEARCY B. SLACK, (Decatur, Ga.), *Eng-.News Record*, Aug. 10, 1933, p. 169-170. Reviewed by N. H. ROY.

A METHOD used by the author for studying the rate of drying or curing of concrete in highway slabs is based upon the hypothesis that the resistance to flow of electrical current in concrete varies with the moisture content, the resistance being high for dry concrete, low for wet concrete. A current is passed from one electrode to another three feet apart on the surface of the pavement. Contact between electrode (a copper plate on one end of a 4 x 4 in. block of wood) and concrete is made through layers of cheese cloth saturated with salt water. Several sections are tested to give an average value. Tests on successive days gave data on the relative moisture content for the same location. The author gives comparative data on three methods of curing.

Stability of straight concrete gravity dams

D. C. HENNY, *Proc. Am. Soc. C. E.*, Sept., 1933, p. 1071. Reviewed by H. J. GILKEY.

THE AUTHOR reviews the changing status of masonry dam design showing how uplift pressure and lowered coefficient of friction from water under the structure introduce factors much more important than the classic middle third conception with uplift neglected. Starting with the thesis that the shearing strength of the masonry is likely to be the critical element in the safety of the structure, the author undertakes to apply, to the problem of the masonry dam, data taken in making compressive tests of stone cores and of concrete cylinders of various strengths and sizes. He points out the lack of published data bearing directly on this subject and states that the experiments cited are too meagre to weigh heavily in selecting safe values in shear.

The problem of uplift pressure is considered briefly as is the related problem of what part of the area of the horizontal cross-sections should be considered as subject to upward pressure (the effective uplift area). The uplift force of water in the pores; the effect of uplift on the stability; and unusual uplift conditions such as would occur with an earthquake disturbance or a badly seamed foundation are also considered. For uplift pressure the Levy requirement is found to be excessive for dams less than 500 ft. high and what are deemed to be more logical requirements are suggested.

The validity of a sliding factor is questioned since a proper safe-guarding against shear on all planes insures safety against sliding on a horizontal plane. This paper relates to gravity dams but the studies can easily be extended to other types of masonry dams.

Photo elastic analysis of stresses in composite materials

A. H. BEYER, Prof. of Civil Engineering, Columbia Univ., and A. G. SOLAKIAN, Lecturer Mechanical Engineering and Research Associate in Civil Engineering, Columbia Univ., *Proc., Am. Soc. C. E.*, Vol. 59, No. 7 (Sept., 1933), page 1121. Reviewed by H. J. GILKEY.

THIS PAPER reports tests which are a continuation of an earlier venture into the field of photo elastic studies of composite materials. (Proc. Am. Soc. C. E., Vol. 58, No. 7, Sept., 1932, page 1248). Such studies are of especial interest to the A. C. I. membership because of the analogy of such materials to reinforced concrete. In the present paper, in fact, the authors have dealt with members purposely designed to parallel the reinforced concrete beam and supply comparisons between the observed and computed stresses. Transparent bakelite (the concrete) is reinforced with aluminum wire (the steel). The bakelite was cast around the aluminum wires to form beams which were subjected to two-point sym-

metrical loading to give pure bending between the load points.

The ends of the rods were provided with sharp and rounded hooks in different specimens and in another case a rod was bent up at about 45 degrees. The favorable distribution of bond stress obtained by using a well-rounded (large radius) bend is shown very clearly in contrast with sharp right angle bends.

The ratio of the modulus of elasticity of the aluminum (9,700,000 p. s. i.) to that of bakelite (670,000 p. s. i. at the temperature used) was about 14.5 which agrees closely with values of n ordinarily accepted for reinforced concrete design. Relatively high initial stresses were present in the aluminum and bakelite which even careful annealing did not eliminate. By observing the initial stresses in the bakelite (no load on the beam) and then observing the final stresses after the beam was loaded, it was possible to obtain the stresses due to the load as a difference. Of course the stresses in the aluminum wires could not be observed optically but they could be computed from the stresses observed in the bakelite. A plain annealed bakelite control member was employed to evaluate the optical effects in terms of stress.

The stresses as determined experimentally were in close agreement with those computed according to the methods followed for reinforced concrete. It needs to be noted, however, that bakelite will take tension and it was not proper, therefore, to follow the usual concrete practice of disregarding tension in the concrete (or bakelite) located below the neutral surface of the beam. The calculations took account, therefore, of the tension present in the bakelite.

The extension of the use of photo elastic methods to composite materials has interesting qualitative possibilities in connection with some phases of reinforced concrete analysis.

Investigations on the nature and sequence of pozzuolanic action

R. Feret, Chef du Laboratoire des Ponts et Chaussees de Boulogne-sur-Mer. Pamphlet issued by the Laboratory—from *Revue des Matereaux de Construction*, Feb. and March, 1933. Reviewed by Stanley G. Cutler.

Discusses the setting effects of various artificial and natural pozzuolans or slags in combination with lime, and deduces a method for calculating and classifying their various activities.

Method of test. From a sample of each slag in a finely pulverized state, its proportions of silica, alumina and iron oxide soluble in hydrochloric acid were obtained. Each was then mixed with an equal portion of lime, the mixture molded and allowed to set three days in moist air, then in water for the various usual setting periods. Each brick was then pulverized to the same fineness as the original slag powder, and the proportions of soluble silica, alumina and iron oxide obtained for the brick in the same manner as for the original slag powder. The increase in these soluble compounds, due to the mixture and to the exposure to water in setting furnishes a measure of the activity of each slag, for which values are deduced.

Study of various slags. The first series of such tests, using several natural cements, baked slags, and pulverized clays result in activity factors for each of these materials. The setting activity of each is inferred to be a factor of the rate of increase in the soluble compounds, different coefficients being used for each. The reason for the increase in soluble compounds in the bricks is due to the greater diffusion of the active slag powders in them.

The results of a second series of tests show the importance of a large propor-

tion of impalpable powder in a pulverized slag, that many slags have properties of delayed activity, greatly modified by air curing. A method is given for deducing the best proportion of slag powder in a mortar or concrete. Other advantages to be gained by admixtures at the expense of strength and activity are discussed.

Study of hydraulic cements. Similar tests to the above, applied to various hydraulic cements, show no such regular increase in soluble compounds as for the slag mixtures, nor is there any discernable relation between such increase and the activity of the cement.

Principles of a theory of the shrinkage of cements*

E. FREYSSINET, *Annales des Ponts et Chaussees*, May-June, 1932. Translated and reviewed by B. MOREELL.

M. FREYSSINET presents a new theory to explain the volumetric changes in concrete. He maintains that volumetric changes which result from the action of setting, from temperature changes, from changes in moisture, from plastic flow and from elastic deformation are inter-related.

He considers concrete to be a porous material which is composed of solid particles, dry voids and spaces filled with entrained water. The spaces filled with water are microscopic. In these spaces, at the surfaces of separation between the liquid and the air there is evaporation, condensation, or equilibrium. In the state of equilibrium, the theory of capillarity permits us to calculate the average dimension of the spaces and also the magnitude of the tension which the liquid exerts on the solid walls by which it is contained. M. Freyssinet has determined the size of the spaces containing water under various hygrometric conditions of the circumambient air. He finds these to vary from 10 to 84 times the diameter of a molecule of water.

The corresponding internal tensions vary from 2140 kilograms per square centimeter (30,400 p. s. i.) to almost zero. These internal tensions are extended in all directions so that the stress conditions which are thus set up in the body are exactly identical to those which would exist if the water were entirely removed and its effect replaced by external pressures in the three principal directions.

Assuming that equilibrium has been established by the internal pressures, when external load is applied the microscopic channels which are partially filled with water are distorted and a new condition of internal tension is set up.

M. Freyssinet by this means explains the phenomena of plastic flow and the volumetric changes resulting from changes in temperature and hygrometric state of the air. He can determine in any particular concrete the number and shape of the interstices and of the particles in between, and as a result he can anticipate all of the peculiarities of concrete.

The foregoing brief outline is intended to give a very general idea of this theory, which is complicated.

*A paper published a year and a half ago and not then reviewed is briefly mentioned now because it is pertinent to current discussions in this JOURNAL. A limited number of translations of the complete article are available for distribution and can be obtained by application to Lieut. Commander B. Moreell, CEC, USN, Bureau of Yards and Docks, Navy Department, Washington, D. C.

A proposed system for the analysis of fresh concrete

W. M. DUNAGAN, (formerly Assist. Prof. Civil Engineering, now Assoc. Prof. Theoretical and Applied Mechanics, Iowa State College), *Bulletin 113*, Iowa Engineering Experiment Station. Reviewed by H. J. GILKEY.

A CRITICISM of concrete as a structural material has been that the quality developed cannot be known until rejection or replacement becomes a major and difficult operation. Within the last ten years techniques have been

evolved for the determination of the approximate proportions of cement, fine and coarse aggregate by analyzing samples of the hardened concrete. This, like strength determinations is in the nature of a postmortem and the information obtained, if adverse, is disquieting but difficult to apply in a manner which will benefit the structure. There has been an increasing recognition of a need for an up-to-the-minute method of inspection by means of which the proportions of the mixture could be verified as it was deposited in the forms.

With usual materials and methods the quality of the hardened concrete is not seriously open to question if the proportions of cement, water and aggregate are right and it has been possible to maintain reasonable assurance of this by the exercise of care in charging materials into the mixer. With the advent of the central mixing plant and other modern developments, it has become increasingly desirable that the purchaser be able to check the quality of the concrete by means of random samples taken at the point of deposit.

Professor Dunagan has taken the lead in working out a technique for the analysis of fresh concrete and has developed equipment to simplify the operations involved. During the last three or four years the so-called "Dunagan Method" and apparatus have been widely used.

The present bulletin is a more or less complete exposition of the method and not only explains in sequence and detail the operations involved, but discusses tolerances and factors affecting the accuracy of the method. Not all who have attempted to use the method have convinced themselves of its dependability and a special effort has been made to point out possible sources of error or difficulty. For example, there may have been instances in which the results of the analysis were more accurate than the actual proportioning, in which case the apparent discrepancy would have been charged wrongly against the method.

References are made to experimental analyses conducted by persons at Lehigh University, U. S. Bureau of Public Roads, Oregon State College, and Iowa State College, and data from the Lehigh University tests are given in connection with the discussion of reasonable tolerances. Definite techniques for the ready determination of specific gravities of aggregates and cement are given and illustrated with numerical examples. An illustrative analysis of fresh concrete is also carried through including the determination of the quantities per cubic yard of concrete, the water cement ratio, and other pertinent items.

The bulletin points out other uses of the analysis of fresh concrete such as for the determination of uniformity obtained under various mixing conditions. This publication contains material that should be of value and interest to practically everyone interested in the proportioning, manufacturing, or placing of concrete.

Demoistured air aids Madden dam cement

Adolph J. Ackerman, *Eng.-News Record*. July 6, 1933, pp. 11-13. Reviewed by N. H. Roy.

The difficult problem of preventing absorption of moisture by cement in transit and temporary storage in the Panama Canal Zone was successfully and scientifically met by the Chief Engineer* for the contractors at Madden Dam. It was necessary to transport the cement from the ship to Madden Siding by rail, thence by

*Adolph J. Ackerman, formerly Chief Engineer, W. E. Callahan Constr. Co., and Peterson, Shirley & Gunther, Contractors, Madden Dam, Canal Zone.

trucks to the dam site, throughout both the dry and the rainy season (72-152 in. annual rainfall). The cement, in moisture-proof paper bags, was transferred from the cars to an inclined belt conveyor. The bags drop onto knives and are slit at four points. A long revolving screen separates the bags and the cement, the cement falling into a conical hopper from whence it is pumped by air to the top of a 6000 bbl. bolted steel silo. As the air is usually highly saturated (often 100%, average 85%), it must be dried before reaching the cement pump. This was accomplished by use of (1) a pipe after-cooler, (2) by condensation in the air receiver, and (3) by a small air separator with automatic water ejector. The cement does not cake in the discharge line. No crust forms in the silo. Standard 10-ton, dump-body, 60 bbl. capacity trucks are loaded from the silo for transfer to the mixing plant silo 13 miles overland. The two silos are similar but not identical. Complete recirculation at the mixing plant prevents the accumulation of aged cement. This method of handling is probably applicable to many jobs in wet or tropical countries. The cement losses are very low.

Concrete caissons sunk by air-lift pumping

Eng.-News Record Aug. 31, 1933, pp. 249-250. Reviewed by N. H. ROY.

A VERY easy and quick method of sinking full size, open, cylindrical, concrete caissons through water, sand and shell beds was employed on the landing jetty at LeVerdon, France. Six sets of ejectors, each composed of a pipe 10 in. in diameter toothed at the bottom for breaking up the sand, and a gas pipe inside for supplying compressed air, comprised the air lift equipment. Each pipe has 36 holes near the bottom for supplying air. A flap valve for equalizing pressures inside and outside the caisson is provided. The air produced an emulsion of sand and water and aerated the mixture and caused it to rise and flow out of the caisson above water level. The caisson was sunk as the sand and shell were removed. The bottoms were concreted under water, the water pumped out of the caisson and concreting proceeded in a dry shell.

Blended cements

RICHARD GRUN and HUGO BECKMANN *Tonindustrie Zeitung*, Vol. 57, No. 70 and 71. Reviewed by INGE LYSE.

A REPORT on the effect of various admixtures in the cement, summarizes the outstanding results as follows:

1. Cements blended with active or inert rock powder are inferior to straight cements, both with respect to strength and durability.

2. This inferiority is due to the fact that the concrete containing the blended cement actually has a lower cement content than the concrete containing ordinary cement.

3. The observed reduction in the lime content for the blended cements is due to the lowering of the cement content by the addition of rock powder. This, however, does not result in a cement with low lime content. Low lime cements are only those which have but little lime in their original ingredients.

4. The durability of the concrete is determined by the density, the cement content, and age, and of the durability of the admixture, provided this takes part in the hardening process. Those admixtures which do not take part in the chemical reactions are merely fillers and blenders detrimental to the product, although their presence tends to reduce the total lime content in the paste.

Current Reviews

Reinforced concrete poles on the continent

ANON. *Concrete Building and Concrete Products.* (Eng.) Serial in July, Aug. and Sept. issues, 1933, pp. 129, 149, 171. Reviewed by J. C. PEARSON.

A brief history of the development and use of concrete poles in Europe, with references to the numerous patents. Data on types, costs, weights, methods of manufacture and erection, loading capacities and tests are given. One of the tabulations gives data on twelve continental high-tension transmission lines carried on concrete poles. Not the least valuable feature of this article is some 20 illustrations of poles in various stages of manufacture, transportation, and erection, also of finished installations, most of which are reproduced from excellent photographs.

Incomplete curing weakens concrete surfaces

WILLIAM J. KREFELD. *Civil Engineering*, Vol. 3, No. 12, Dec. 1933. Reviewed by J. R. SHANK.

Tests made in Columbia University laboratories show that the escape of water from the top surface of poured concrete causes the upper layers to be materially reduced in strength and abrasive resistance, and that surface treatments to prevent this are worth while, applied immediately after placing. About 40 per cent of the total water lost during 60 days of exposure evaporated the first day, and about 65 per cent had disappeared at the end of three days.

A mortar block, of 1:2½ proportions, 6.1 gal. of water per sack of cement, 18 in. deep by about 24 in. dia. was cast in a metal mold, top open, and cured at 71°F. and 69 per cent rel. hum. for 28 days. Cores one inch dia. were cut and sections from various depths were tested in compression. The cores from the top were about half as strong as those in the lower 12 in. Top cores from 6 in. cylinders 8 in. high showed 50 per cent more wear in the Dorry abrasion testing machine than those from the mid-depth. Mortars surface treated and untreated were exposed for 60 days and analyzed for water loss. Untreated mortars lost 59 per cent of the total mixing water whereas those treated lost only 18 per cent.

Advancements with light weight, aerating concrete

K. R. MULLER. *Zement*, Vol. 22, No. 45, p 631-633, 1933. Reviewed by INGE LYSE.

Porenbeton is an aerating concrete produced by means of hydrogen peroxide and chloride of lime. The unit weight may be as low as 30 lb. per cu. ft. for insulation and about 75 lb. per cu. ft. for construction purposes. The concrete for insulation has a compressive strength of about 150 p.s.i. at 28 days; voids are about 75 per cent and the mortar mix about 3 to 1—320 kg. cement, 105 kg. sand, 5.4 kg. hydrogen, and 170 kg. water. The mix for construction purposes consists of 250 kg. cement, 900 kg. sand, 3.5 kg. hydrogen, and 200 kg. water, and the product has a compressive strength at 28 days of approximately 350 p.s.i. The Porenbeton is used principally for building blocks and precast slabs.

Penetration test for workability

"Der Eindringversuch zum Messen der Verarbeitbarkeit des Betons." OTTO GRAF, Stuttgart. *Beton u. Eisen*, Vol. 32, Heft 20, Oct. 20, 1933, p. 321. Reviewed by A. A. BRIELMAIER.

In the early days of reinforced concrete there was little attention to workability. The shortcomings of the slump and spread tests as gauges of workability lead the author to bring forward

a test, in line with suggestions which were perhaps premature when he made them in 1909. His recent investigation, supported by the German Reinforced Concrete Committee, resulted in the design and construction of a device for penetration tests. In the device, a tripod supports a bullet shaped plunger (from photograph, about 4 in. diam. and 12 in. long) eight inches above the concrete, which is in a container about 12 in. square and 12 to 20 in. deep. The plunger is dropped into the concrete.

The new apparatus brings out more correctly the variation in workability between mixes with gravel and those with crushed rock, while disagreeing more or less with the results of the spread test. Concrete made with gravel shows a smaller penetration than crushed rock concrete, although the two mixes have the same spread. The penetration and spread tests agree for sandy gravel mixes.

The penetration test for workability requires further study to determine whether it is suitable for all conditions.

Preservatives for concrete

Staff Article in *Das Betonwerk*, Vol. 21, No. 49, 50, 51, 1933. Reviewed by INGE LYSE.

Our country is not the only one producing all kinds and types of perservative and protective materials for concrete. The listgiven in this article, of admixtures, surface treatments and surface coatings available on the German market, reaches a total of 341, with names from *Abernol* to *Zimmerit*. An introductory statement points out the great variety of recommended practices for the different materials and also the general lack of knowledge of their actual qualities. This list of concrete preservatives is therefore given as an aid in selecting the proper material for any given condition. A brief description of the claim for, and the application of, each material is generally included.

Vibrated concrete

H. DUHRKOP. *Ingenioren*, Nov. 18, 1933. Reviewed by INGE LYSE.

Vibration in placing concrete is gaining ground rapidly in Europe also and this paper discusses the use of external, as well as internal vibrators with illustrations of equipment used in Denmark. The effect of vibration on such qualities as strength, weight, consistency, cement content, and other items are discussed on basis of the most recent American papers on this subject. The author concludes that the favorable results obtained on vibrated concrete point towards a change from hand tamping to mechanical placing in ordinary concrete construction.

Concrete electrically heated

"Die elektrische Beheizung des Betons." ANDREAS RETHY, Moscow. *Beton u. Eisen*, Vol. 32, Heft 18, Sept. 20, 1933, p. 282. Reviewed by A. A. BRIELMAIER.

Electrical heating of concrete for winter construction is the subject of considerable interest. The Standardization Division of the II Construction Trust, Moscow, has developed a heating method based on the use of the concrete itself as a highly resistive conductor, instead of only heating the surfaces. Three points were kept in mind throughout the investigation: ordinary job procedure must not be changed; a simple electrical arrangement must insure the heating of even large masses under adverse conditions; the quality of the concrete must not suffer.

A three-phase, alternating current at 110 to 220 volts was used. Electrode grids of sheet metal are sufficient for thin slabs, while scrap reinforcing rods serve as electrodes for heavier sections. The first are placed on the concrete; the second are embedded in the structural member with an end projecting out of the forms. The best heating procedure was found to be: increase the concrete temperature by 9° F. per hour

until 122° F. is reached; hold this temperature until the concrete is 24 hours old; raise the temperature to 158° F.; hold this until the age is 36 hours and shut off the current.

This heating system was employed in the construction of an industrial structure involving 260 cu. yds. of concrete. The air temperature was 1° F. at the beginning of pouring, and varied between 14° F. and −13° F. during the process. No special covering or protective measures were adopted. A daily concrete production of 24 cu. yds. was attained. Sections of 3 to 4 cu. yds. were heated at a time, requiring about 1½ hours per section. The consumption of current varies from 61 kwh. to 115 kwh. per cu. yd. Control tests indicated that the above described process results in about 70 per cent of the designed strength. Eight-inch cubes, cut out of the structure after 28 days of thawing weather, developed a strength 1.2 that of the control cubes. This may be due to the higher temperature as well as the effect of the current on the chemical reaction of setting. Further investigation is expected to shed more light on this point, and on the influence that electrical heating may have on the tensile strength and on the elasticity.

The early date at which the forms are stripped increases re-use. This taken into account, the heating cost varied between 2½ and 13 per cent of the total concrete cost.

It is planned to employ this heating system in the construction of shafts for the Moscow subways. These shafts are to be sunk in artificially frozen river sand.

Iporit light-weight concrete

Walter Hahn. *Das Betonwerk*, Vol. 21, No. 44, p. 441-443, 1933. Reviewed by Inge Lyse.

Among the numerous aerated light weight concretes used in Germany, Iporit is a new type produced by introducing a powder into the fresh mortar. The strength of the Iporit concrete is higher for high-early-strength cement than for ordinary portland cement. Fine sand should be used as an increase in fineness decreases the weight of the product. The consistency of the mortar is liquid with a net water content of about 50 per cent of the weight of the cement. The weight of the Iporit is between 63 and 88 lb. per cu. ft. The compressive strength of the ordinary 1:3 mortar mix is about 350 p.s.i. Test results indicate that the heat insulating quality of Iporit was nearly four times that of ordinary gravel concrete.

The rupture of concrete and the estimation of working stresses

Lionel Norman Jamieson, *Journal of the Institution of Engineers Australia*, Vol. 5, No. 10, Oct. 1933. Reviewed by J. R. Shank.

The author brings to view many fundamentals of mechanics not ordinarily mentioned in treatises on concrete and reinforced concrete. He calls attention to the nature of failure of concrete test specimens; that the pyramid or cone generally considered to indicate a pure compression failure results from the fact that the frictional resistance between the concrete and the testing machine parts is greater than the lateral expansion indicated by Poisson's ratio. "The normal compression failure in concrete is by splitting across planes parallel to the axis of loading. There is a moderately sound theoretical basis for the assertion that, of all types of stress and strain (shear stress and strain, compressive stress and strain, tension and extension), the tendency to rupture is independent of all except *tension* (tensile strain). Rupture is a consequence of excessive extension only." The proper bases for considering ultimate strength of concrete are elasticity, Poisson's

ratio and the limit of extensibility. An accurate method of measuring Poisson's ratio is indicated. The limit of extensibility can be found from Poisson's ratio and the ultimate compression strength. The role of reinforcement is discussed in which he says: "A steel tension member embedded in concrete is, in general, more extensible than the steel member alone." A concept for the function of spiral steel is given.

Tests on a reinforced-concrete arch of the Arlington Memorial bridge

CYRUS C. FISHBURN and JOHN L. NAGLE. Research Paper No. 609, *Bureau of Standards Journal of Research*, Vol. 11, No. 5, p. 567, Nov. 1933. Reviewed by D. E. PARSONS.

The paper gives the results of measurements of temperatures and deformations in one of the reinforced concrete arches of the Arlington Memorial bridge at Washington, D. C. Measurements were made during the placing of the concrete in the arch and at intervals thereafter for more than two years.

The span under test was a solid barrel, open spandrel arch having a clear span of 177 ft. and a clear rise of 36.6 ft. The upstream and downstream edges of the barrel supported heavy spandrel walls of granite ashlar. Cross walls served to distribute the loads caused by the granite facing and to support the roadway. The structure was of unusual design in that the edges of the wide (90 ft.) arch barrel supported the heavy spandrel walls. The piers were of mass concrete about 35 ft. wide and 45 ft. high.

In a portion of the barrel which was 4.8 ft. thick, the temperature of the concrete increased from 73° F. when placed to about 143° F. 24 hours later. Nearly normal temperature was reached in about 15 days. The increase in temperature was much less for thinner portions and where the thickness was 2.25 ft., nearly normal temperature was attained in eight days.

After the concrete had cooled, the average temperature closely approximated the average of the air temperature during the previous five days. The maximum observed average temperature of the arch barrel was 85° F. and the minimum was 24° F. The maximum and minimum air temperatures during the same period were 106° F. and 7° F., respectively.

The deflections of the arch were affected measurably by the restraint of the superstructure; this restraint reduced by about 13 per cent the crown deflections caused by uniform changes in temperature. Unequal vertical deflections in transverse sections of the arch barrel occurred with changes in temperature because of differences in physical properties of the granite facing arches and the concrete arch barrel. The effect of shrinkage and flow on the crown deflection during the period from the eighteenth day to the twenty-fourth month after completion of the first two arch barrel strips was equivalent to a drop of 27° F. in average arch temperature.

The upper surfaces of the piers were warped, apparently by changes in temperature.

Also, the piers rotated with changes in the temperature of the arch and variations of the load. The maximum observed rotation between consecutive tests was about 40 millionths radian. The settlement of the piers was unequal the difference sometimes being as much as 0.15 inch.

Plotting charts for columns in compression with tension over part of section

ODD ALBERT, *Civil Engineering*, Vol. 3, No. 2, Dec. 1933, pp. 667-9. Reviewed by J. R. SHANK.

This paper deserves more attention than is usually given to just another man's idea of the one best chart for

solving problems in reinforced concrete analysis and design. The author shows by means of an example how charts are designed and the result illuminates the method. It is a three-part chart with straight line curves for solving for the percentage of symmetrically placed steel required for a member such as an arch ring carrying bending and direct stress when part of the section is under tension. The author has published many other charts of noteworthy character.

Reinforced concrete railway viaducts near Belfast

R. L. M'Ilmoyle, *The Structural Engineer* (England), Vol. 11, No. 11, p. 430-443 (Nov. 1933). Reviewed by V. P. Jensen.

The project is of especial interest since it shows the influence of C. S. Whitney's report, "Plain and Reinforced Concrete Arches" (Journal, American Concrete Inst., March, 1932, *Proceedings*, Vol. 28, p. 479), and W. M. Dunagan's, "A Method of Determining the Constituents of Fresh Concrete" (Dec., 1929, Vol. 26, p. 202).

The job involves a main viaduct about 630 ft. long and a secondary viaduct about 400 ft. long. The main structure consists of 3 open spandrel parabolic arches with 89 ft. span and 30 ft. rise, flanked by small arches on either side. The secondary viaduct, employing similar arches, passes under the main viaduct. Dimensions of structural parts, details of reinforcement, and construction methods are given.

A feature of the construction was the striking of the steel three-hinged arch centers 4-days after the last section had been poured. At this time the crown section, the first poured, was 9 days old. A maximum deflection of ⅛ in. was observed in the large arches at striking and no further deflections could be detected as the concrete aged. Dunagan's method of analyzing fresh concrete was commended.

Effect of vibration and delayed finishing on the quality of pavement slabs

F. H. Jackson and W. F. Kellermann. *Public Roads*, Oct., 1933, Vol. 14, No. 8, pp. 129-154. Reviewed by Authors.

This report summarizes the results of two series of tests, in one of which a study was made of the effect of finishing pavement concrete with a power finishing machine equipped with vibrators mounted on both front and rear screeds and another where a study was made of the effect of finishing delayed to remove excess water—the Johnson Method* patented.

The general procedure which applied to both series of tests was as follows: A pavement 9 ft. wide was constructed using the conventional type of batching plant, paver and finishing machine. It was divided by headers into slabs 9 ft. long, each section later being subdivided into four slabs 27 in. wide by 5 ft. long for test purposes. Curing consisted of 24 hour burlap followed by a 2-in. earth covering which was left on the pavement, being wet down for 10 days after placement and for 14 days prior to testing.

The pavement slabs were tested for flexural strength in the field, after which cores were drilled from the broken slabs and crushing strength, density and absorption tests run. Wear tests were also made on the broken slabs. Parallel strength tests were made on control beams and cylinders made during construction.

Aggregates used were: 2 fine, a coarse river sand, fineness modulus 2.98 and a fine sand from the same source fineness modulus, 2.13; 3 coarse, a river gravel, a crushed limestone and a blast furnace slag weighing 83 lb. per cu. ft. dry rodded. These gave six combinations of materials. For each combination of coarse aggregate with coarse sand a base proportion was set up having a cement factor of 6 sacks per cu. yd. When fine sand was substituted for coarse sand the proportion of coarse aggregate was held constant and the sand content reduced to take care of difference in grading, the result being a reduction of 0.1 to 0.2 parts of sand by volume, depending upon the coarse aggregate.

The procedure followed for each combination of materials was to cast six sections in one day. The first section was base proportions, 2-in. slump, finished in the usual manner with a

**Proceedings*, Amer. Concrete Inst., Vol. 23, 1927, p. 458. "Time as a Factor in Making Concrete Pavement".

power screed. Five additional sections were constructed, all vibrated and all somewhat drier than the base mix. In two of these proportions were the same as the base mix and the drier consistency obtained by reducing the water content until slumps of approximately 1 in. and ½ in. were obtained. In the other three sections the water-cement ratio was the same as the base mix and the drier consistency obtained by adding ¼, ½ and ¾ parts, respectively, to the coarse aggregate content. This constituted one round of tests and, with the exception of slag with fine sand, two such rounds were run, making a total of 66 sections.

In the second series slag aggregate was not included but in its place a sand-gravel mixture conforming in grading to Platte River gravel, maximum size ½ in. The base proportions used with gravel and stone in the first series were duplicated in addition to a leaner mix containing one-half part more fine sand and one part more coarse aggregate than the base mix. The Platte River gravel was proportioned on the basis of one part cement to four and four and one-half parts aggregate by dry-rodded volume.

The procedure followed in the second series was to cast one section of each of the above combinations of materials and proportions using a 2-in. slump and finishing by the standard method. For each of these sections a duplicate was cast, using approximately a 3-in. slump and finishing by the Johnson method. Two rounds of tests were run, making a total of 40 sections.

Briefly, the Johnson method of finishing consists of screeding by hand, floating with a long-handled wooden float, rolling off the excess water with a light weight sheet metal roller, and belting. A sand-cement mixture (1:1 by volume) is then applied at a rate of one sack of cement per 20 sq. yds. of pavement and this is belted and followed at intervals by floating until no more free water is brought to the surface. The total lapsed time between placement and final finish varies from 2¼ to 3¾ hours. No machinery is employed in the finishing operations, all work being done by hand.

In both series of tests the workability of the concrete was judged by the amount of honeycombing at the section of failure and on the bottom of the individual slabs.

CONCLUSIONS

The major facts developed by this investigation are summarized as follows:

Effect of Vibration

1. When finished by the standard method, the minimum slump required to insure pavement slabs substantially free from honeycomb was found to be approximately 2½ in.

2. When finished by vibration, the minimum slump required to insure the same degree of uniformity was found to be approximately 1 in. (In the previous investigation it was found that standard-finished pavement slabs constructed of concrete having a 1-in. slump were less uniform in quality than similar slabs in which 2-in. to 3-in. slump concrete was used.)

3. The average flexural strength of all vibrated pavement slabs in which the concrete showed an average slump of 1 in. was somewhat higher than the average strength of the standard-finished concrete of the same proportions in which the average slump was 2 in. (In the previous investigation standard-finished slabs constructed of 1-in. slump concrete were generally lower in flexural strength than similar concrete having a 2-in. slump.)

4. The average increase noted under (3) was due to the marked increase in strength for the group of sections in which crushed stone was used as coarse aggregate. For the sections containing gravel as coarse aggregate, the vibrated concrete having a 1-in. slump showed a lower average flexural strength than the standard-finished concrete having a 2-in. slump. In the case of slag, the strengths for the two slumps were about the same.

5. The average flexural strength of all vibrated pavement slabs in which the concrete showed an average slump of ½ in. was lower than the average strength of the standard-finished concrete of the same proportions in which the slump averaged 2 in.

6. The average decrease noted under (5) was due to the marked decrease in strength for the group of sections in which gravel was used as coarse aggregate. In the case of stone and slag the average strength of the vibrated con-

crete having a ½-in. slump was about the same as the average strength of the standard-finish sections having a 2-in. slump.

7. The average flexural strength of all vibrated pavement slabs constructed of concrete having the same proportions of cement, fine aggregate, and water but containing one fourth part more coarse aggregate by volume than the standard-finished concrete, was considerably higher than that of the standard-finished concrete. (In the previous investigation, concrete containing more coarse aggregate by volume than the base mix when finished by the standard method showed lower flexural strengths than the base mix, the decrease being roughly proportioned to the amount of coarse aggregate added.)

8. The increase noted under (7) was most marked in the case of the crushed stone concrete and least in the case of the gravel concrete.

9. The average flexural strength of all vibrated concrete slabs containing one half part more coarse aggregate than the standard-finished concrete was somewhat higher than that of the standard-finished concrete. The sections containing crushed stone gave considerably higher values, the sections containing slag slightly higher, and the sections containing gravel lower.

10. The average flexural strength of all vibrated concrete slabs containing three fourths part more coarse aggregate than the standard-finished concrete was lower than that of the standard-finished concrete. The sections containing crushed stone and slag gave about the same results, whereas the sections containing gravel showed a decrease.

11. For concrete having the same water-cement ratio, finishing by vibration did not increase the crushing strength of cores drilled from the pavement slags containing gravel and slag as coarse aggregate. In the case of crushed stone a considerable increase in crushing strength was noted for the vibrated concrete containing one half and three fourths parts additional coarse aggregate.

12. The process of finishing by vibration did not adversely affect the hardness of the surface.

Effect of Delayed Finishing by the Johnson Method

1. Pavement slabs finished by the Johnson method showed higher crushing and flexural strengths than standard-finished concrete of the same proportion but mixed with less water. This observation applies to both of the proportions studied.

2. Pavement slabs finished by the Johnson method developed substantially the same crushing and flexural strengths as standard-finished slabs containing one half part less fine aggregate and one part less coarse aggregate. The Johnson-finished slabs, after correction for the amount of cement in the "dry mix," contained approximately one sack of cement less per cubic yard of concrete than the standard finished slabs.

3. Although, for each proportion, the water-cement ratio at the time of mixing was lower in the case of the standard-finished concrete, the final density of the concrete as revealed by tests on the cores was higher in the case of the Johnson-finished slabs.

4. Variations in crushing and flexural strength due to changes in proportions and methods of finishing were more marked in the crushed stone concrete than in the gravel concrete.

5. The process of finishing by the Johnson method did not adversely affect the hardness of the concrete.

6. The process of finishing by the Johnson method resulted in decreasing the void-cement ratio approximately 0.1 more than the corresponding decrease shown for the standard-finish concrete.

Colored surfaces for artificial stones and cement products:

WILLI SERKIN. *Tonindustrie-Zeitung*, Vol. 57, No. 99, p. 1168-1171, 1933. Reviewed by INGE LYSE

Describes types of coloring materials for stone and concrete products, in groups including waterglass products, lime products, oil and resin varnishes, cellulose varnishes, heated varnishes, and asphalt paints. In recent years colored metallic waterproofing coatings which make visible the structural nature of the artificial stones have gained much ground. Among these the fluo-silicate of copper gives a blue-green color, the fluo-silicate of iron yellow-brown, and the fluo-silicate of lead with a weak solution of potassium bichromate, a bright yellow color. The problem of colored surfaces is so difficult that it should be left to the specialists for successful solution.

A. S. T. M. Tentative specifications

Tentative Specifications for Ready Mixed Concrete (C94-33T).

Tentative Methods of: Test for Absorption by Aggregates for Concrete (Laboratory Determination) (C95-33T) and Field Test for Absorption of Mixing Water by Aggregates for Concrete (C96-33T) were recently accepted by the American Society for Testing Materials for publication as tentative on the recommendation of its Committee C-9 on Concrete and Concrete Aggregate. These are now available for comment and criticism before final adoption.

Current Reviews

Thin self-carrying shell vaults and their computation

R. VALETTE, *LeGenie Civil*, Vol. 104, No. 4, p. 85-88, Jan. 27, 1934. Reviewed by R. L. BERTIN.

THE author reviews the development of a system of shell vaults used for the first time in Germany in 1924. He describes this system as consisting of a thin vaulted shell with vertical sides bearing on edge beams of considerable magnitude and limited transversely by means of solid diaphragms. He points out that this type of construction does not constitute the original application of vaults utilized as carrying beams, and cites several structures built in France from 1910 on in which more and more of the shell was assumed to act integrally with the supporting beams and the beams themselves progressively reduced to a size just sufficient properly to encase the reinforcing bars.

A comparison of the multiple vault system used in Germany and the large vault system favored in France is given and the features in favor of the latter are stated as follows:

(1) A reduction in the developed surface of the shells.

(2) A reduction in the magnitude of the ties and ribs as compared with the edge beams.

(3) An improved air circulation and larger spans.

(4) Simplification of erection through the use of rolling scaffolds.

The author critizes the principles involved in the design of shell vaults with vertical sides. He shows graphically the deformation of a vault model with a span equal to four times the rise supported on solid and diaphragms under various types of loading, and concludes therefrom that if the vertical sides of the vault are effectively free, there is evidently no reaction along the edges but there is a displacement of these sides, as well as of the vault and correlative transverse flexure of the vault. He points out that when the vertical sides become anchored to the edge beam and the shell of the adjoining vaults, the vertical sides become fixed and the shell is subjected to transverse flexure just as a vault supported along its edges would be.

The rest of the treatise is devoted to the development of basic principles applying to the design of vaults of large spans and reduced spans, with and without ribs, and loaded uniformly or with concentrations.

Reference is made to the following issues of *Le Genie Civil* in which the subject of shell vaults is discussed. June 9, 1928, p. 564; Feb. 2, 1929, p. 113; Feb. 12, 1927, p. 172; June 11, 1932, p. 592.

Results of tests on the effect of molasses on concrete

MILES N. CLAIR AND M. A. MORRISSEY (Vice-President and Testing Engineer, respectively, The Thompson-Lichtner Co.), *Engineering News Record*, Dec. 28, p. 775. Reviewed by N. H. ROY.

IT IS generally known that molasses and other classes of sugar often have a deleterious effect upon concrete structures. This is especially true when sugars are present in fresh concrete. In many such cases concrete does not harden.

The authors report on results of tests made with standard portland cement briquettes immersed in two grades of molasses, light and dark. Some sets of briquettes were immersed in molasses at the ages of 1, 7 and 28 days; others were given protective coatings at the age of 7 days and then immersed in light molasses.

The specimens immersed in molasses at early ages and subsequently broken had the appearance of having been frozen. The surfaces of these specimens were soft and honey-combed. Cracks also were found as deep as the molasses had penetrated. Twenty-eight day specimens were not greatly affected by the molasses. Light, refined molasses had greater effect upon the briquettes than the dark molasses.

Protective coverings on briquettes used in the tests were effective in preventing the destructive effects of the molasses for a few months. After one year of exposure, only one of the three protective coverings used in the tests remained effective. Sodium silicate covered specimens showed an improvement over the untreated specimens of the same age.

The authors believe that the action of the molasses on the cement results in the formation of calcium succinate, which disrupts the structure of the cement.

Recommendations made by the authors for treating concrete structures which are to be exposed to molasses are: (1) Treat the surface at an age of 3 to 5 days with three coats of sodium silicate, the first two coats using the commercial water glass diluted to half its concentration, and the last coat used full strength, after which allow 28 days for concrete to harden. (2) If the structure *must* be exposed to molasses at an early age, apply a heavy coating of sodium silicate to the surface. This seems to be the most effective method investigated.

Widening and strengthening a cast iron bridge with concrete

LeGenie Civil, Vol. 104, No. 5, p. 101, Feb. 3, 1934. Reviewed by R. L. BERTIN.

THIS bridge, known as Heliopolis Bridge, is on the national highway between Bône and Constantine in Algeria. The span is 55 metres and the cast iron arches have a rise approximately 1/10 of the span. The bridge, built about 70 years ago, was no longer capable of carrying the ever-increasing traffic and was reinforced with concrete as follows:

The dead load was reduced by removing the roadway and building a light one way lane above the level of the old deck.

New reinforced concrete arches were built on each side of the cast iron ones. While the cast iron ribs were restrained, the new reinforced concrete arches were designed as three hinged arches to eliminate the high stresses which expansion would cause in arches of constant sections with a rise less than ⅙ of the span.

To eliminate the interference of the cast iron sections at the hinges, they were cut at these points when the concrete arches were completed.

The arches at the crown were enlarged horizontally and stopped about 12 in. each side the axis; jacks were used to pre-stress the arches and the gap finally filled with concrete. The deck, also of reinforced concrete was then installed. The advantages of this method were saving of scaffolding, maintenance of traffic and speed of erection.

Design and construction of composite slab and girder bridges

ALLEN WALTON KNIGHT, *The Journal of the Institution of Engineers Australia*, Vol. 6, No. 1, Jan., 1934, p. 10. Reviewed by J. R. SHANK.

SHORT span highway bridges made up of longitudinal rolled steel beams carrying a continuous concrete slab are so designed as to use the concrete slab as the compression flange and the steel beams as the tension flange of the bridge as a whole. The steel beams are not encased. Stirrups are welded to the top flanges to develop the large amount of horizontal shear imposed. The steel beams are propped up before the concrete slab is poured to produce

a camber, the amount of which has been carefully calculated, so that when the concrete is hardened and the props are removed the combination of initial stresses and bending stresses causes approximately uniform tension across the beam cross-section and uniform longitudinal compression in the slab. Careful calculations are made to investigate and provide for the effect on the slab of variable deflections in the steel beams due to concentrated live loads. The effects of volume changes of the concrete such as plastic flow, shrinkage, etc., are investigated and provided for in the design. The paper as a whole is very thorough and goes into considerable detail.

Tests on two-way slabs

W. Gehler and H. Amos, *Bulletin No. 70, German Reinforced Concrete Committee.* Reviewed by Inge Lyse.

The purpose of the investigation reported in this bulletin was to establish the most favorable dimensions and systems of reinforcement for two-way slabs, freely supported. A few supplementary tests were made on plates with anchored corners and plates which were anchored along the full length of support. The report gives detailed information on the method of making and testing and presents all the test data. It contains 182 pages, including 97 figures and 99 tables. The results of this extensive investigation have been summarized as follows:

1. All of the two-way slabs acted as isotropic plates up to the load at which the first crack appeared. The type and amount of reinforcement did not take part at these low loads. The deflection increased in accordance with the straight line relationship.

2. The resistance to the occurrence of cracks is about three times greater for freely supported two-way slabs than it is for reinforced concrete beams. The load at first crack for slabs was considerably greater than the working load, the ratio for the freely supported slabs being 2 to 1, while for beams the corresponding ratio has been found to vary from 0.3 to 0.7.

3. A rather complicated stress distribution occurs after the first cracks have appeared. Further study of this problem is necessary for the final solution.

4. The ultimate factor of safety, that is, the ratio between the ultimate load and the design load, varied between 4 and 5, being relatively high and particularly higher than for beams where the factor varied from 2½ to 3.

Slabs anchored to the supporting beams received so much benefit from this rigidity that such slabs showed a high factor of safety without having special torsion reinforcement.

Reinforced concrete design

Le Constructeur de Ciment Arme. Reviewed by P. H. Bates.

The reviewer does not find it possible to present in the form of reviews any particular paper which has appeared in this publication in the last few months (July, 1933 to Jan., 1934 inclusive). This publication, however, would be of much interest to the designers of reinforced concrete in that monthly it presents mathematical discussions relating to reinforced concrete design. It is believed that citing some of the topics which have been discussed will be of value to the membership of the Institute. It should be borne in mind that the papers cited, in practically no case, go into construction methods, but are devoted to the mathematical development of the design of the types of structures indicated in the titles:

"Large cylindrical reservoirs."

"Calculation of continuous beams."

"Calculation of balcony beams."

"Calculations of hyperstatic systems."

"A new point of view on the classification of isostatic and hyperstatic

systems from the modern viewpoint of strength of materials."

"Analytical calculations of columns under the action of horizontal forces."

"Calculation of maximum moments in floor slabs."

"Calculations of octagonal columns in eccentrically loaded rigid frames."

"Unequal continuous beams resting freely on their supports."

"Encased metal beams (from the new regulations for the use of reinforced concrete of Hungary)."

"Tables for the solution of arch construction."

"Note on the bending moments in the elements of superimposed multiple arches."

Most of these papers extend through two or three numbers. There has also been extending throughout this period a report on some conferences on reinforced concrete held at the Conservatoire National des Arts et Metiers. This has covered a wide variety of subjects. One that has been particularly stressed is that of concrete piles.

Strength of reinforced concrete slabs

OTTO GRAF, *Bulletin No. 73, 1933, German Reinforced Concrete Committee.* Reviewed by INGE LYSE.

A REPORT on tests of strength of reinforced concrete slabs with concentrated load near one support and on tests of strength of concrete at the point where reinforcement is bent up in reinforced concrete beams.

The first group of tests consisted of three reinforced concrete slabs with cubes and plain concrete beams as control specimens. The slabs failed underneath the load, a section being pushed through. The results showed that the smaller the distance from the support to the concentrated load, the greater was the maximum load carried. The strength of the slab increased with the increase in cement content in the concrete, but this increase was less than for the cubes. The strength of slabs subjected to concentrated loads cannot therefore be obtained directly from the cube strength. Neither the flexural nor the shearing strength bears a direct relation to the strength of the slab.

The second group of tests consisted of 14 reinforced concrete beams. The bent-up bars had different radii of curvature. It was found that the strength of the beam (in diagonal tension) increased with the increase in radius of curvature of the bent-up bars. For ordinary conditions the radius of the curvature should be at least five times the diameter of the bent-up bar. The resistance of the concrete to the splitting action of the bent-up bars increased much less than did the compressive strength of the concrete.

Tests on brick masonry beams

M. O. WITHEY (Professoı of Mechanics, University of Wisconsin, Madison, Wis.), *Proceedings, A. S. T. M.*, Vol. 33, Part II, p. 651. AUTHOR'S SYNOPSIS.

DATA are given on the shear and bending strengths of twenty-five 8 by 12-in. reinforced brick beams tested under third-point loading over an 8-ft. span. Three widely different varieties of brick, and several variations in percentage of longitudinal steel between 0.5 and 2.3 per cent, as well as different percentages of stirrup reinforcement were used. The tests indicate that a high degree of flexural strength and shear strength can be developed in reinforced brick beams provided due attention is paid to mortar bond, coursing, amount, and arrangement of reinforcement, and filling of joints. Formulas for reinforced concrete design with appropriate constants can be used to calculate stresses and deflections of reinforced brick beams.

Surface scaling of concrete pavements on Illinois state highways

From an unpublished report of the Bureau of Materials, Illinois Division of Highways, Nov., 1933. Reviewed by Highway Research Information Service.

This report contains an analysis of data on surface scaling on concrete pavements placed during 1932. Similar analysis was made in an earlier report for the years 1930 and 1931. The purpose of the scale surveys is to learn under what circumstances the scaling occurs and to determine by statistical analysis the factors which appear to influence the scaling.

The information collected for scaled areas includes:

a. Accurate location of scale with an estimate of amount in lineal feet of pavement.

b. Class of scaling; "thin" if appearance of slab is not impaired, "medium" if casually noticeable but not exposing coarse aggregate, "heavy" if it exceeds the medium in thickness and appearance.

c. Source of all materials.

d. Date and approximate hour of the day when built.

e. Air temperature coincident with construction.

f. Climatic conditions coincident with construction.

g. Climatic conditions, if unusual immediately before and after construction.

h. Opinion as to whether brooming was properly done.

i. Details regarding all finishing and curing, time of beginning and progress of scaling.

j. Data in detail for each scaled area, length, width of slab, class and amount of scale, etc.

k. Any specific information on unusual circumstances during construction.

From the studies, the tentative theory is advanced that the immediate cause of scaling is crystal growth. The slab consists of the concrete covered with a thin layer of relatively porous mortar of inferior strength, which in turn is covered with a relatively impervious film of neat cement. Under certain favorable conditions, dissolved salts are carried to the surface mortar and deposited at planes of cleavage or lamination. Some of the more important factors seem to be climatic conditions, kind of cement, time of day when laid. Brooming, when properly done, seems to be effective in preventing light scale but may not prevent medium and heavy scale.

The yardage of pavement actually scaled in Illinois is very small, 0.43 per cent, and probably does not affect the structural life of the pavement. However, the problem becomes serious in some sections where heavy scale occurs, principally because of its effect on the appearance.

Concentrated loads on slabs

Clyde T. Morris, *Bulletin No. 80*, Ohio State University Engineering Experiment Station. Reviewed by J. R. Shank.

The bulletin gives a brief history of the experimental investigations of the subject and a review of the theoretical treatment published by Dr. H. M. Westergaard in *Public Roads*, Mar.

SUMMARY OF DATA COLLECTED FROM SCALE SURVEY OF PAVEMENTS CONSTRUCTED ON STATE BOND ISSUE ROUTES DURING THE 1930, 1931, AND 1932 CONSTRUCTION SEASONS

Year of Construction	Lin. Ft. of Pavement in Terms of 18-Ft. Width of Slab: Length Pavement Built (1)	Length of Pv'mt. Showing Scaling (2)	Amount of Scaling: Total (3)	Distribution: Thin	Medium	Heavy	Per Cent of Surface Showing Scaling (2)x100 / (1)	Per Cent of Surface Actually Scaled (3)x100 / (1)	Intensity of Scaling (3)x100 / (2) (Per Cent)
1930	4,225,053	475,975	22,835.3	13,611.0	3,503.9	5,720.4	11.27	0.54	4.80
1931	3,893,841	347,010	19,018.4	14,883.9	2,055.9	2,078.6	8.91	0.49	5.48
1932	4,196,345	165,781	17,957.7	3,552.7	5,107.0	9,298.0	3.95	0.43	10.83

1930. The various experimental data are plotted and correlated with the theoretical equations of Doctor Westergaard and simple rules for the design of slabs subjected to concentrated loads are developed. The following is quoted from the conclusions.

"From the foregoing discussion the following rules for design may be accepted as safe and not greatly in error. When results are desired which are theoretically more exact, the designing moments may be taken either from the curves presented or from Westergaard's theoretical equations. The physical properties of reinforced concrete are not sufficiently uniform to warrant more refinement in making calculations for design than is given by the proposed rules."

Calorimeter installation for studies of heat generation in mass concrete

S. B. BIDDLE, JR. AND J. W. KELLY, *Proceedings, A. S. T. M.*, Vol. 33, Part II, p. 571. AUTHOR'S SYNOPSIS.

OUTSTANDING features of the cement investigation for the Boulder dam are the measurement of the heat generated by 93 cements during hydration and the curing of specimens for strength and volume-change tests under variable temperature conditions corresponding to those in mass concrete. The system of calorimeters and curing chambers employed at the University of California is described in this paper.

Direct measurements of temperature rise in mass concrete are made for certain cements in six adiabatic calorimeters, and indirect measurements of heat generation of cement are made for all cements in a heat-of-solution calorimeter by dissolving neat-cement samples in acid. Approximate curing temperatures corresponding to those of mass concrete are determined in advance by measuring temperature rise of concrete specimens in high-insulation calorimeters. The curing is then carried out in variable-temperature chests, controlled by specially designed instruments.

The comparative advantages of the different methods of determining heat generation are herein pointed out, and the function of each method in the coordinated program of cement investigation is shown.

Improving concrete quality by paying bonuses

M. H. KLEGERMAN, *Engineering News Record*, Oct. 12, 1933, p. 434-437. Reviewed by N. H. ROY.

A VERY unusual and apparently satisfactory method of specifying and paying for about 50,000 cu. yd. of concrete was employed by a group of municipalities in New Jersey. Much of the concrete in sewers and disposal plant construction was placed under difficult conditions such as deep trenches, quicksand, tunnels, railroad and river crossings, and special structures. Concrete exposed to sewage and sewer gas more often fails by disintegration due to lack of density than of strength. Where concrete is subjected to wetting, drying, freezing, and thawing, density is considered one of the important factors. Strength, imperviousness and density were desired in this project. Specifications required that strengths of various classes of concrete must not fall below a minimum for that class. A mix for each class of concrete was suggested; the proportions were optional to a great extent with the contractor. Water-cement ratio for 28 day strength, characteristics of the cement to be used, and slump limits were additional controls specified so as to obtain a durable and close-textured concrete. Bids were received on the basis of the various strengths classified and a clause provided for the payment of a bonus for strength in excess of the base strength specified. The bonus

per cu. yd. was 25 cents for each 100 p. s. i. above the base for concrete placed between April 15 and Oct. 15 and 50 cents for concrete placed between Oct. 16 to April 14. Maximum payments were placed at 500 and 600 p. s. i. above base. This method of bonus payment together with the unusual specifications resulted in extra compressive strength and a more watertight and durable concrete. The plan was satisfactory to the contractors also. The usual strength determinations were made on 6 x 12-in. cylinders and occasionally on cores from the structure. Bonuses were paid on the basis of cylinder strength. Considering all contracts, the average bonus was about 16 cents per cu. yd. per 100 p. s. i. excess strength, while for some concrete not on the bonus basis the cost per 100 p.s.i. excess strength was about 38 cents.

Experimental cement plant at University of California

J. W. Kelly (Engineer Mat. Lab., University of California), *Engineering News Record*, Nov. 22, 1933, p. 519-521. Reviewed by N. H. Roy.

A dual-kiln unit and a three-stage continuous grinding mill have been constructed at the materials laboratory. This equipment, together with the laboratory facilities, provides for a study of cement problems from the raw material to the final product.

The effects of heat treatment of clinker in controlled atmospheres on the formation of clinker compounds is one of the interesting studies made. Problems in connection with lightweight aggregate manufacture are under investigation. A study of limes is contemplated. The fineness of ground cement is accurately determined.

The grinding mills are used to reduce raw material or clinker. The temperatures of grinding are controlled within 10 degrees of the desired heat. The same hood system is used for cooling or for heat temperatures up to 400°F.

Both kilns are 30 ft. long, 30 in. outside diameter and are operated under automatic control. One of the kilns is used for burning clinker; the second for heat treatment of the burned clinker under oxidizing or reducing conditions.

The construction of grinding mills and kilns and their operations are given in detail. The method evolved by the laboratory for determining the fineness or specific surface of cement is illustrated.

The durability of cement mortars

C. A. Hughes, *Proceedings, A. S. T. M.*, Vol. 33, Part II, p. 511. Author's Synopsis.

The paper includes data on the absorptions, compressive strength, resistance to sodium sulfate and to three artificial cycles of freezing and thawing of mortars made from 18 brands of portland cement and 10 special cements. An attempt is made to show the relations, if any, existing between the above properties of the mortars. In addition, the relation of mortar durability as measured by immersion in 10 per cent sulfate or artificial freezing-and-thawing cycles to the calculated constitution and to Merriman's index of disintegration is discussed.

New Italian cement specifications of 1933

Zement, No. 3 and 4, 1934. Abstracted by Inge Lyse.

These specifications divide the hydraulic cementing materials into three major groups:

a. Limes (hydraulic lime and strong hydraulic lime).

b. Cements (portland cement, aluminum cement, slag cement and pozzuolanic cement).

c. Other cement materials (rapid binding and slow binding).

For the cements both tensile and compressive tests are required on specimens made from the 1:3 mortar mix having standard sand. The test specimens are stored in water until day of test. The minimum strength requirements are as follows:

1. Portland cement, slag cement and pozzuolanic cement:

Tensile	25 kg/cm^2	(355 p.s.i.)	at 7 days
strength and	30 kg/cm^2	(425 p.s.i.)	at 28 days
Compressive	350 kg/cm^2	(5000 p.s.i.)	at 7 days
strength	450 kg/cm^2	(6400 p.s.i.)	at 28 days

2. High early strength portland cement, slag cement and pozzuolanic cement:

Tensile	20 kg/cm^2	(285 p.s.i.)	at 3 days
strength	30 kg/cm^2	(425 p.s.i.)	at 7 days
	35 kg/cm^2	(500 p.s.i.)	at 28 days
Compressive	250 kg/cm^2	(3550 p.s.i.)	at 3 days
strength	450 kg/cm^2	(6400 p.s.i.)	at 7 days
	600 kg/cm^2	(8500 p.s.i.)	at 28 days

3. Alumina cement:

Tensile	25 kg/cm^2	(355 p.s.i.)	at 24 hr.
strength	30 kg/cm^2	(425 p.s.i.)	at 3 days
	40 kg/cm^2	(570 p.s.i.)	at 28 days
Compressive	300 kg/cm^2	(4250 p.s.i.)	at 24 hr.
strength	500 kg/cm^2	(7100 p.s.i.)	at 3 days
	650 kg/cm^2	(9250 p.s.i.)	at 38 days

4. Other cementing materials:

Tensile	18 kg/cm^2	(255 p.s.i.)	at 7 days
strength	22 kg/cm^2	(310 p.s.i.)	at 28 days
Compressive	180 kg/cm^2	(2550 p.s.i.)	at 7 days
strength	300 kg/cm^2	(4250 p.s.i.)	at 28 days

These new specifications became effective January 1, 1934.

Behavior of reinforcement in concrete of various compositions

OTTO GRAF, *Bulletin No. 71, 1933, German Reinforced Concrete Committee.* Reviewed by INGE LYSE.

THE report, covering investigations carried out at the Government Testing Laboratories at Berlin-Dahlem, deals principally with questions raised in connection with the new regulations for reinforced concrete design. Among subjects studied was the necessary cement content in the concrete for the protection of the reinforcement against rusting. The results showed that the specified cement contents gave satisfactory protection. Studies were also made on the absorption and the unit weight of concrete of different compositions, and on the consistency of concrete as measured by a penetration apparatus. The results showed the flexural strength as well as the compressive strength of concrete increased in direct proportion to the increase in cement content for any given gradation of aggregate. The unit weight of the concrete showed considerable irregularity and varied between specific gravities of 2.00 and 2.52. The penetration apparatus is of a very simple construction, consisting of a 3.95-in. diameter cylinder with a spherical end. This cylinder weighs 13¼ lb. and is dropped a distance of about 8 in. The amount of penetration of the cylinder into the concrete is taken as a measure of placeability. It is concluded that this penetration apparatus is better suited for concrete control than is the ordinary flow table.

Effect of adding a siliceous material to portland cements

R. W. CARLSON AND G. E. TROXELL, *Proceedings, A. S. T. M.*, Vol. 33, Part II, p. 484. AUTHOR'S SYNOPSIS.

THIS paper points out the possible advantages to be derived from the blending of siliceous materials with portland cements, particularly with regard to mass-concrete construction. The results of a series of tests to determine volume changes, compressive strength and heat of hydration of blended cements are presented.

The variables in these tests are calcination temperature of the silica, chemical composition of the cement with which the silica is blended, curing condition, method of blending, and proportions of silica and cement. It is found that these variables determine to a considerable extent whether or not the blending of silica with cement leads to favorable or unfavorable properties of mortar.

In general, volume changes of mortar due to loss of moisture are shown to be lower at the early ages and higher at the later ages for blended cement than for plain cement. The tendency of the silica to increase the contraction at

later ages appears to be least for the high-lime cement, for preliminary curing under mass-concrete conditions, and for silica calcined to incipient fusion.

The 7-day compressive strength of mortar is shown to be lower for all blends of silica and cement than for plain cement, but the strengths are more nearly equal at later ages. As much as 30 per cent of silica may be blended with a high-lime cement without appreciably reducing the mortar strength at the age of 3 months.

It is shown that the effect of blending silica with cement is to reduce the heat of hydration, but not by the full percentage of cement replacement.

It is concluded that the test results show sufficient promise to justify further investigation, covering a wider range of materials and conditions of blending, together with a study of additional properties such as economy of cement manufacture, and workability and durability of concrete. (See p. 369, this JOURNAL—EDITOR.)

Participation of slabs in the resistance of reinforced concrete beams

HENRY LOSSIER, *LeGenie Civil,* Vol. 104, No. 5, p. 108-112, Feb. 3, 1934. Reviewed by R. L. BERTIN.

THE author calls attention to the fact that this problem which originated with the concept of reinforced concrete is still the object of much discussion. He reviews the principal regulations promulgated in France and elsewhere as follows:

Notation

l = span of beam

e = spacing of beams (center to center)

t = thickness of beam

h = overall depth of beam

d = thickness of slab

b = the permissible width of slab participating in the resistance of the beam including the stem of the beam

e' = clear distance between beams $= (e - t)$

$b' = b - t$

France

(1) *La Circulaire du Ministre des Travaux Publics,* Oct. 20, 1906.

$$b \leq \frac{l}{3} \quad b \leq .75\,e$$

No limitation of b with reference to d being given.

(2) *La Commission du Ciment Arme*

$$b = e\left(1 - \frac{e^2}{l^2}\right)$$

$\frac{e}{l}$ being limited to 0.5

Mesnager

$$b = \frac{e}{1 \div 3\left(\frac{e}{l}\right)^2}$$

Regulations of 1930 LaChambre Syndicale des Constructeurs en Ciment arme of France

$$b = \frac{e'}{\sqrt[3]{1 \div \left(\frac{3e'}{l}\right)^3}} + t$$

Germany—Rules of 1916

$$b \leq 8t \leq 16d \leq 4h \leq e$$

The smallest of these values controlling. Note that no limitation with reference to the span is given.

Rules of 1925

$b \leq 12d \div t \div 2g$ in which g is the extreme width of fillets between the slabs and the beam

and $b \leq \frac{l}{2}$

Rules of 1932 same as those of 1925.

Switzerland

Rules of 1909

$$b \leq \frac{l}{4} \leq 20d.$$

Rules of 1915

$$b \leq t \div \frac{l}{5} \leq t \div 16d.$$

The new regulations

$$b \leq t \div \frac{l}{4} \leq t \div 16d \leq e$$

Tests conducted by LaCommission du Ciment Armé of France in 1902 led to the conclusion that the ratio of b to l should not exceed .36 to .385.

The tests of Schüle published in 1909 led to the conclusion that this ratio should not exceed .25.

Tests by the author led to values between the two above mentioned.

The paper continues with a discussion of the influence of each variable on the value of b and concludes as follows:

(a) Neither the value h nor t need be considered in determining or limiting b'.

(b) The value b' should be equated to the span l and e' rather than expressed by independent limits.

(c) A safe value of b in terms of d is expressed by $b \leq 18d \div t$.

(d) For continuous beams the effective span may be taken as $.75l$ and l for simple beams.

(e) In the case of shell vaults with ribs in compression along their full length, it may be considered that the entire slab between ribs is effective.

(h) The slabs across the beams should be thoroughly reinforced top and bottom to resist the stresses due to continuity and the transverse tensile forces tending to separate the slab from the beam.

(i) The minimum thickness of slabs should be 4 cm for floors with hollow fillers and 6 cm for solid slabs.

Strength of concrete in view of new research

ADALBERT POGANY, *Zement*, No. 4 and 5, 1934. Reviewed by INGE LYSE.

VERY small test specimens were investigated by means of a microscope. Particularly shrinkage crack formations were given close observations and the following conclusions were reached regarding the factors which control these cracks:

1. The amount and the quality of the cement.
2. The size of the concrete specimen.
3. The moisture and temperature conditions during the testing period.

With respect to the strength qualities it is stated that besides the strength quality of the cement, the gradation of the aggregate, the method of placing, etc., the strength is to a large extent controlled by the conditions of the hardening period. The shrinkage cracks are particularly harmful to the strength of the concrete and when these were eliminated, strengths of such high salue as 1000 kg. per cm^2 (14,000 p. s. i.) were obtained.

Behavior of concrete protective agents with respect to the effect of active liquids

H. BURCHARTZ AND H. W. GONELL, *Bulletin No. 72, 1933, German Reinforced Concrete Committee.* Reviewed by INGE LYSE.

FOUR different types of protective agents were used, namely:

1. Admixtures which were incorporated in the concrete, mostly as additions to the mixing water.
2. Absorptive agents which should have the ability of hardening the surface of the concrete.
3. Surface coatings which as a protective cover were supposed to prevent penetration of harmful liquids.
4. Coatings, such as oils, which are supposed to be beneficial to the concrete.

Concrete specimens protected with 38 different materials were exposed in: (a) tap water, (b) 5 per cent solution of magnesium sulfate, (c) 5 per cent solution of sodium sulfate, (d) 0.5 per cent solution of sulphuric acid, (e) machine oil, and (f) linseed oil.

It is concluded that none of the protective agents showed any beneficial effect on concrete exposed to magnesium sulfate and sodium sulfate solutions, certain of them were even found to be detrimental. The admixtures to the concrete which were supposed to prevent or decrease the absorption of the active liquids showed no such behavior. The change in weight was just as great as for the specimens with no admixtures.

The so-called absorptive agents showed little or no improvement in the resistance of concrete toward the action of active liquids, while all the surface coatings showed a definite improvement. However, they cannot completely prevent the penetration of the active liquids during a long time of exposure. Of the two types of oil coatings used, one showed beneficial effect, while the other one showed no effect.

Concrete rigid frame bridges for railroad loadings

D. A. McGavern (Designing Engineer, Cincinnati Union Terminal Co.), *Engineering News Record*, Nov. 2, 1933, p. 526-528. Reviewed by N. H. Roy.

An unusual type of structure for railroad loadings eminently practical for the location for which it was designed and probably applicable to other locations but not to all locations, is clearly described in the above paper.

The larger of two similar bridges has a total length of 328 ft. and was designed to carry 23 tracks into the Union Station in Cincinnati. The structure is composed of nine distinctly separate two-span, barrel-type, concrete, rigid-frames each about 35 ft. wide. The rigid-frames are hinged at the side and center wall footings due to prohibitive cost of construction of footings which would prevent rotation. The deck slab, side and center walls are continuous.

Frames were designed for Cooper's E-65 live load, impact, horizontal earth pressure, live-load and dead load surcharge, traction, temperature changes, and settlement of foundations. The general shape of the frame cross section was determined largely by the moment and shear requirements as limited by clearance lines. Due to high shear from heavy live loads the thickness of the slab near midspan could not be reduced as in the case of the ordinary highway bridge. All frames are skewed, more or less.

A frame section was obtained by approximate methods, analyzed, and used as a basis for a final section. Various methods of analysis for this type of indeterminate continuous structure were employed including two variations of the slop-deflection equations, the graphical solution by A. Strassner, and the method of distribution of moments by Hardy Cross. Each method was quickly applied and results checked very well.

An approximate analysis was made to determine the effects of skew and compensated for by using transverse bars in the slabs.

Influence lines were drawn for corner and intermediate moments. Moments were also determined for uniform horizontal load against each side wall separately, a triangular load against each side wall, and a horizontal load applied at the center of the slab. Combinations of these moments were made and maximum positive and negative moment and shear diagrams were drawn from which concrete stresses and steel areas were calculated. In combinations of these moments and shears, including settlement of any one foundation, stresses 50 per cent higher than normal were allowed. This condition governed only for positive steel in the slab near the center pier and side walls.

The effect of vertical settlement of one foundation was to increase the stresses in the frame and to reduce the load on that foundation. Horizontal

movements of the footings were considered. Footings were designed to resist such movements.

Side walls were designed for direct and bending stress combined. Negative steel in the outside faces of the side walls was carried, by long bends, into the top of the slabs and anchored by hooks or long embedment with the negative steel of the slab. The width of the side wall at the top of the footings was small hence closely-spaced ties were employed.

Concrete strength requirements were 2500 p. s. i. at 28 days for all parts of the structure except the lower two feet of the sidewalls and center pier which was 3000 p. s. i. minimum. High early strength concrete was not justified unless a higher unit stress was allowed.

It is suggested by the author that in similar designs a checker follow immediately behind the designer up to the point of determining the elastic properties of the structure and solving for the redundants.

Due to continuity of walls and slab in this type of structure there is a saving in materials over other types of structures and is worthy of consideration under similar conditions.

Crack and rust developments in reinforced concrete members

LOTHAR KRUGER, *Bulletin No. 71, 1933, German Reinforced Concrete Committee.* Reviewed by INGE LYSE.

THE investigation was carried out at the Government Testing Laboratories at Berlin-Dahlem. The results are summarized as follows:

1. The protective cover is of great importance for the behavior of the steel. Steel having thin protective cover was much more affected than was steel having a thick cover. The reduction of the cross-sectional area of the steel was in all cases so small as to be of no practical importance.

2. The concrete containing 400 kg. cement per cubic meter aggregate (7¼ U. S. sacks per cu. yd.) protected the concrete much better than did the concrete having cement contents of 300 and 250 kg. The leaner the concrete mix, the more extensive was the rusting of the steel.

3. The wetness of the concrete had a large effect upon rust protective quality. Even a small addition of water to a moist earth-consistency concrete gave, within certain limits, a definite effect upon the rust protection.

4. The type of cement had no effect upon the behavior of the steel.

5. The steel was more affected in beams stored with no protection than in beams well protected during the storage period. This was particularly noticeable for steel having only 1 cm (0.4 inch) thick concrete cover.

Status for specifications for hydraulic cements in the United States

P. H. BATES (Chief, Clay and Silicate Products Division, U. S. Bureau of Standards), *Proceedings, A. S. T. M.*, Vol. 33, Part II, p. 462. AUTHOR'S SYNOPSIS.

THIS paper is a discussion of the many uses of hydraulic cements and the many service conditions which they must meet. From a consideration of these the question is raised as to the possibility of any one type of cement being able to meet adequately the various demands. The properties of the various types are then presented with a discussion of their adaptability to various services, using as a particular example the conditions which had to be met in the construction of Boulder Canyon Dam and the nature of the cement specified therefor.

The outstanding requirements of the present portland cement standards of the Society are presented and some discussion of their significance is made. There is also given a brief historical summary of all the revisions of the

Society's standards for this commodity, with some explanation of why some of the revisions were made.

Attention is also directed to the tendency of a part of the public to propose standards deviating from those of the Society. Several reasons are advanced for this state and it is suggested that Committee C-1 on Cement recognize the need of several kinds of cement for the various uses and also the inadequacy of some of the specified test methods.

Thin section concrete arches as built in Switzerland

M. S. Ketchum, Jr., *Engineering News Record*, Jan. 11, 1934, p. 44-45. Reviewed by N. H. Roy.

The writer calls attention to a type of bridge that has been in use in Europe for about 40 years but not known in this country. It has been used chiefly for steel bridges but since 1925 reinforced concrete bridges have been built with very thin arch ribs and posts. These ribs are flexible and designed to take direct thrust only. Unbalanced live load moments are resisted by deep, stiff, concrete girders which also carry the floor and railings. Several bridges, designed and built in this manner are giving satisfactory service.

Improving concrete by means of "pervibration"

I. Lebelle, Administrateur-délégué de la Société Les Procédés Techniques de Construction, *LeGenie Civil*, Vol. 104, No. 1, p. 21-22, January 6, 1934. Reviewed by R. L. Bertin

"Pervibration" is a term copyrighted by the above named society at Paris which designates the process of internal vibration covered in various countries by patents owned by the Society.

This article is a discussion of a paper by H. Lossier, published in *Le Genie Civil* of Nov. 4, 1933, p. 445.

The author answers the criticism of Mr. Lossier to the use of pervibration, namely that concrete is a poor medium for the transmission of vibration and that while it increases the strength when applied to dry concrete it reduces its resistance by as much as 10 per cent when applied to wet concrete because of induced segregation.

The author states that these conclusions were derived from tests conducted at the Laboratory of the Technische Hochschule of Stuttgart reported by Otto Graf and Kurt Wals in *Beton und Eisen*, Aug. 20, 1933, p. 252. He points out that these tests were conducted on specimens containing too high a percentage of fine aggregate and too wet to bring out the real advantages of internal vibration. He contends and cites the paper of T. C. Powers (Journal, Amer. Concrete Inst., June 1933, *Proceedings*, Vol. 29, p. 373), in support of his contention that the chief advantages to be gained by internal vibration is an improvement in uniformity and the reduction of the water content and therefore of cement for a concrete of given strength made possible through the use of coarser graduation or a reduction in the proportion of fine to coarse aggregate.

Apparatus for fabricating and testing mass-concrete cylinders

R. F. Blanks and E. N. Vidal, *Proceedings, A. S. T. M.*, Vol. 33, Part II, p. 538. Author's Synopsis.

This paper describes equipment and apparatus for fabricating and testing mass concrete cylinders of various sizes up to 36 in. in diameter in a comprehensive program of mass-concrete research for Boulder dam. Special methods of laboratory procedure developed for conducting large cylinder tests are discussed and test results indicating the degree of uniformity secured are presented briefly in comparison with those obtained in usual series of laboratory tests programs.

Hydrostatic analogy for arch structures

H. M. WESTERGAARD (Prof. of Theoretical and Applied Mechanics, University of Illinois) *Engineering News Record*, Dec. 28, 1933, p. 788-789. Reviewed by N. H. ROY.

THE author presents in detail a method of determining moments and deflections of arches by considering a piece of wood floating in water. The "floating arch" and the actual arch have the same center line shape but in case of the floating arch in the water it lies in a horizontal plane. The top and bottom of the floating arch are flat; cross sections are rectangular with constant depth but variable width. Stresses, deflections, and influence lines may be determined in arches or beams. This excellent method of analysis is closely related to and supplements the column analogy introduced by Hardy Cross (University of Illinois Eng. Exp. Sta., Bull. 215). These methods should be of value to designers of concrete and other structures.

Current Reviews

Highway and hydraulic construction during 1933

T. Hokerberg, *Teknisk Tidskrift,* No. 3, p. 25, Mar. 1934. Reviewed by Inge Lyse.

This article presents an interesting review of new construction in Sweden during 1933. The bridge construction was particularly active and a number of reinforced concrete bridges are described. Hinged frames with up to 30m (98 ft.) free span and hinged arches with up to 44.7m (147 ft.) free span were built in reinforced concrete. Rigid arch construction was used for spans up to 76m (250 ft.). Reinforced concrete arches with steel beams as tension ties for the horizontal thrust have been used quite frequently with spans up to 71m (233 ft.). These tie-rod arches are build in flat country with the roadway directly on the tension beams. The slender sections of these bridges will make the American engineer wonder why similar designs cannot be used in this country. High early strength cements were used in certain bridges with resulting concrete strength as high as 5700 p.s.i. at four days. The article contains a number of photographs of the larger bridges.

37th Convention of the German Concrete Institute

Tonindustrie-Zeitung, No. 30 and 31, 1934. Reviewed by Inge Lyse

The 37th annual convention of the German Concrete Institute was held Apr. 5 and 6, 1934, with about 500 in attendance, a smaller number than in previous years. Dr. Petry, the Secretary, gave a detailed report of the work of the Institute during the preceding year and Dr. Otzen, the President of the Berlin-Dahlen laboratory presented a paper on the value of tests of materials. Among the contributions: Professor Graf on problems of present and past researches in reinforced concrete; Professor Kleinlogel on the usefulness of a special type of wire mesh as reinforcement; Mr. Ehlers, questions arising in reinforced concrete foundation work; Dr. Rüth on structures to resist bombardments by aeroplanes (an illustrated lecture); Dr. Prüss, a report on new types of concrete structures; Mr. Baritsch, problems encountered in deepening harbors; Mr. Sehl, on concrete highways and Dr. Schneider-Arnoldi, a report on practical experiences with concrete houses particularly light weight concrete; Dr. Mast, reinforcing the foundation for a railroad bridge in Berlin; Professor Neuffer, advancements solid bridges. The convention closed with two papers on the construction of the Mosel bridge at Koblenz (the Adolf Hitler bridge). Mr. Holzmann discussed the practical and Professor Gehler the technical problems of this structure.

Comparison of the value of coverings used to protect concrete during the setting and hardening period

Edmond Marcotte, Chef du Laboratoire d'Essais physiques et mecaniques des Ponts et Chaussées, *LeGenie Civil,* Vol. 104, No. 4, p. 91-92, Jan. 27, 1934. Reviewed by R. L. Bertin.

The author at the beginning coins the French word "cure" to define what is known in this country as curing.

He points out that much more attention is paid to curing in the United States than in France which he attributes to the much greater spread of temperature.

He describes the tests conducted at Arlington by the Bureau of Public Roads where a large number of protective coverings were investigated,

also the more recent tests of the same bureau reported in *Public Roads* of July 1933, in which cotton padding was used. The conclusions of the Bureau are given and the author's own conclusion is that from these tests, three days' curing seems sufficient. However, he recommends longer periods, particularly if the protective medium is not as efficient as that used in the United States.

Old Ohio road shows record service

F. H. JACKSON (Senior Engineer of Tests, Bureau of Public Roads), *Engineering News Record*, Sept. 28, 1933, p. 378-380. Reviewed by N. H. ROY.

A PLAIN concrete pavement about 20 miles long built in 1914-15 of good aggregates has successfully withstood heavy traffic with very little replacement and with only the usual maintenance. Surveys made in 1924 and 1932 show high compressive strengths and moduli of rupture from cores and beams of both gravel and crushed stone concrete. The compressive strengths were about equal for the two classes of aggregates but the modulus of rupture for the gravel concrete was higher than for crushed stone concrete notwithstanding the fact that there were more cracks per slab (especially transverse cracks) in the gravel concrete. The crack chart is very interesting. Assuming a general relation between flexure and tension, the larger number of cracks would be expected in the concrete of lower flexural strength, but the reverse was noted. That property of concrete which prevents or retards cracking is apparently not that of strength.

Rapid method for the determination of the specific surface of portland cement

L. A. WAGNER, *Proceedings, A. S. T. M.*, Vol. 33, Part II, p. 553. AUTHOR'S SYNOPSIS.

AN apparatus and method are described for making rapid determinations of the specific surface and also the particle size distribution of portland cement. The apparatus, which is essentially a turbidimeter, consists of a source of light of constant intensity which passes through a suspension of the cement in kerosine and then into a photo-electric cell. The current generated in the cell is measured with a microammeter and the readings afford a measure of the turbidity of the suspension. The relation between the turbidity of the suspension and the surface area of the suspended particles is calculated. Particle-size distribution is obtained by observing changes in turbidity as the particles settle from the suspension. Details of construction and operation of the apparatus are given in an appendix to the paper.

Thermal movements in dams

PAUL JOYE, *Le Ciment*, 39th Year, No. 4, p. 85-91, 1934. Reviewed by P. H. BATES.

THIS paper is one of those presented at the Congress on Dams at Stockholm in 1933. The author, a Swiss engineer, cites some of the studies made on dams in Switzerland on the heat developed during making and on the cracking which has resulted. A brief discussion of the apparatus used is given. This is followed by a general discussion of movements in the dams.

The temperature attained in jobs is greater than that indicated by laboratory tests. Thermal equilibrium is generally attained only after many years. The Barberine Dam attained a temperature of 30° C. and cooled but 6° annually. For a long period the center was 30° higher than the exterior Under the action of the higher temperatures the center of the mass hardens more rapidly and attains a much greater modulus of elasticity than the exterior. He states that some means suggested for lowering temperatures may accomplish this result but may

cause more unfavorable effects, as for instance, more water in the mass will reduce the temperature but will also reduce its tensile strength and increase the shrinkage due to drying.

The author also believes that in general, physical methods of lowering the temperature should be preferred over those designed to influence the chemical phenomena of setting. The first method suggesting itself is that of using blocks of moderate dimensions during construction and particularly limiting the height of any one casting of a block, and furthermore, exposing their surfaces to the air for cooling. The use of certain physical methods cooled the Spitallam Dam very rapidly. A thermometer in the center of the large mass recorded cooling from 37° C. to 21° C. from October to December and reduction four months later to 9° C.

The author also invites attention to the rise in temperature in concrete adjacent to construction joints which may be brought about by placing new concrete in the joints. He finally cautions that thermal dilation of concrete is not nearly as marked as hygrometric dilation. The coefficient of dilation due to drying may be as much as forty times as great as the coefficient of thermal dilation.

Building Research Board recommendations for a code of practice for the use of reinforced concrete in buildings

Department of Scientific and Industrial Research, London. Reviewed by F. E. Richart.

This report of the Reinforced Concrete Structures Committee of the Building Research Board of a recommended code of practice for the use of reinforced concrete in buildings has departed in several ways from current rules of English practice. Changes include the omission of all but essential formulas, the addition in appendices of standard methods of tests, and the provision for certain types of work under special permission.

Three sets of working stresses for concrete are proposed, depending upon three degrees of control and inspection. Concrete of Ordinary Grade is that placed without rigid inspection or tests; that of High Grade calls for preliminary tests, for regular inspection and control tests; while concrete of Special Grade, with careful design for a desired strength, placed under rigid inspection and control is rewarded by the assignment of comparatively high working stresses. The permissible concrete stresses, in terms of the strength, f, of job control cubes, are $\frac{f}{3}$ for flexural compression, $\frac{4f}{15}$ for direct compression, $\frac{f}{30}$ for shearing stress and $\frac{f}{30} + 25$ for bond; with the further limitation that the last two shall not exceed a value of 150 p.s.i. Values of f provided for range from 2250 to 4688 p.s.i. on control cubes. Steel stresses of 18,000 and 20,000 p.s.i. are permitted in flexural members and 13,500 and 15,000 p.s.i. in columns. A special provision is made for hard drawn steel wire used in slabs.

The general requirements for workmanship in reinforced concrete compare well with current American practice.

In design, attention has been given to the effects of shrinkage and creep of concrete and design rules were chosen to produce safe construction when these effects are present. For beams and slabs, leading conditions to be met in design are defined, and the designer is allowed some leeway in reducing the theoretical negative moment by providing a corresponding increase in positive moment. The

only moment coefficients specified for continuous beams and slabs are the time-honored values $\frac{W}{10}$ and $\frac{W}{12}$, which are given "for cases of uniform loading with approximately equal spans, unless more exact estimates are made." The design of two-way slabs is treated very simply by means of one diagram and a set of moment coefficients. On the subject of shear and bond, the usual equations are given for shearing stress, bond stress and stress in vertical stirrups. Other provisions are in accord with American practice.

The section on column design seems to reflect recent developments in America. The modular ratio, n, has been eliminated from the column formulas and a direct expression for stress in vertical and spiral steel substituted. For spiral columns, the equation contains a term for the effect of the spiral (Considere type) and the same factor of safety is used for both tied and spiral columns. The use of very large spiral percentages is allowed in some cases, and the permissible load on columns with heavy spiral reinforcement may reach a value equal to the gross area of the concrete section times one-half the cube strength (or about 0.6 times the cylinder strength) plus the area of the vertical steel times 15,000 p.s.i. In such a column, as much as two-thirds of the safe load may be attributed to the spiral reinforcement. The reduction in strength of long columns is similar to, but less conservative than, the reduction provided in our Joint Committee specifications. A method of computing bending moments in exterior columns is given. The section on flat slabs is similar to that of the Joint Committee, though less detailed.

There are nine appendices to the report giving general building clauses and various detailed methods of test for reinforcing steel, concrete materials, for consistency and strength of concrete.

The application of high-grade steel in reinforced concrete

FRITZ VON EMPERGER. *The Structural Engineer* (England), Synopsis in Vol. XI, No. 12, Dec. 1933, p. 475-477, Lecture in Vol. XII, No. 3, Mar. 1934, p. 160-183. Reviewed by V. P. JENSEN.

A FRESH analysis of design practice in reinforced concrete and a new attempt to understand the behavior of reinforced concrete structures has been stimulated by the use of certain types of high-yield-point steel, notably wire netting and "Isteg" steel. Isteg, successfully developed in Austria, is a type of reinforcement made by twisting upon one another two mild steel rods, at the same time keeping the ends of the rods a constant distance apart and producing thereby a considerable permanent stretching.

Opinions and conclusions are based on an extensive series of tests made by the German and Austrian reinforced concrete committees and by various universities in Central Europe. A large number of these tests are described and their results given. In general, the members tested were rectangular beams, T-beams, and reinforced concrete tension specimens. Steel percentages ranged up to approximately 2.7 per cent, steel yield-points varied between 37,000 and 60,000 p.s.i., and the concrete strengths between 2,100 and 4,200 p.s.i.

It is shown that the steel yield strength, and not the concrete, is the limiting factor in determining the failure of beams reinforced with percentages up to 1.5 of mild steel and 1.3 of high-strength steel. For beams reinforced with mild or high-strength steel first cracks were observed on initial loading at approximately the permissible loadings of rectangular beams and approximately one-half the permissible loadings of T-beams. These

cracks are considered unavoidable and to be distinguished from "dangerous" cracks which occur when the bond is broken and which expose the steel to corrosive elements. Dangerous cracks were obtained with plain bars at a steel stress of 28,000 p.s.i. and with deformed rods at stresses between 40,000 and 50,000 p.s.i.

Conclusions are as follows:

(1) Given the existing qualities of building materials, given the fact that concrete is not taken full advantage of, and that we have so far no steel to match the concrete, I strongly recommend that we should not impose on the use of concrete more restrictions than are necessary, and should restrict especially concrete compression stresses only in constructions which are liable tò collapse on account of concrete compression stress, which is not the case for ordinary percentages, as I have proved.

(2) With regard to cracks, since we have no very strong steel on the market today, the best steels having a yield point of scarcely 60,000 lb. per sq. in., in the inteiests of reinforced concrete we ought to avail outselves of what we have got, and should not restrict the permissible stresses to less than half the yield point, whatever that may be, provided we are satisfied by scientific investigations that at these stresses there is no danger of laying the steel bare.

*Concrete electrically heated for Moscow subway**

A. Rethy, *Beton u. Eisen*, V. 33, No. 4, p. 58-60, Feb. 20, 1934. Reviewed by A. A. Brielmaier.

The two kilometer Mjasnizka Radius portion of the Moscow Subway project lies in a limestone, just under a 98 ft. stratum of quicksand. In places, the tunnel roof extends into this quicksand. The danger to adjacent buildings caused by pumping out the large amount of seepage, led to the adoption of the freezing method for sinking Shaft No. 20 (Aug.-Sept., 1933). The shaft, 18.5 ft. in diam. and 95 ft. deep, was excavated and braced without further difficulty.

The air temperature in the shaft was 32° F. and at the earth face; 14° F. The use of heated concrete materials was rejected because of the probability of bad effects due to non-uniform cooling of the concrete. This problem was solved by heating the concrete electrically.

The progress schedule called for a ten foot height of the concrete ring, amounting to 19 cu. yds. This mass was electrically heated each day. A unit of three cubic yards was heated through one circuit. Eight circuits were controlled at a central switchboard and again, at each section heated. Six of these were replaced each day. The electrodes were inserted in the concrete after every meter of pour. The 24-hour heating period required a current consumption of 38 KWH per cu. yd. At the point under treatment, the shaft temperature was at 90° F., while in the, as yet unheated, portions above, it was 37° F.

The ratio of the heating cost to the total cost of the concrete was 7 per cent.

Cube tests indicated that if the heating is maintained for 24 hours immediately after placing, the 3-day strength is 95 per cent of that of the 28-day strength of untreated cubes. On this project a puzzolan-portland cement (30 per cent puzzolan earth; 70 per cent clinker cement) was used.

This is the first instance of heating concrete by this method under job conditions. A total of 157 cu. yds. was electrically treated. On the same project, in the course of the present year, it is planned to heat 26,000 cu. yds. of concrete in the same manner. A book on the subject of heating concrete by electrical methods, is in preparation by Mr. Rethy, in German.

Vibrated concrete pavements

F. V. Reagel, John W. Kushing, F. H. Jackson, and W. F. Kellermann, *Engineering News Record*, Apr. 26, 1934, p. 528-531 and R. R. Litehiser, C. W. Allen, V. L. Glover, R. B. Gage, Lion Gardiner, *Engineering News Record*, May 3, 1934, p. 561-7. Reviewed by N. H. Roy.

Valuable papers on the subject of vibrated concrete for pavements. Tests conducted by the engineer-authors represent highway pavement in several states. Reports of tests are by and

*Elektrisch beheizter Beton beim Bau der Moskauer Untergrundbahn.

from engineers of New Jersey, Michigan, Bureau of Public Roads, Ohio, Illinois and Missouri.*

The practical ideas underlying the development of the use of vibrated concrete for pavements are (1) to use dry mixtures, 1 in. slump or less, (2) to keep workability that favors uniformity and solidity, (3) to reduce costs by reduction of cement content or by use of leaner mixes, (4) to permit present day speeds of paving.

The tests reported show or summarize data from pavements, (1) poured and finished in the standard or usual manner with standard equipment, (2) dry and leaner mixes, spread by hand, puddled, finished by special vibratory machine, (3) by use of dry and lean mixes with complete coordinated plant.

Generally speaking the results of the use of vibrated concrete in the several states have been, (1) the economic and mechanical practicability of compacted concrete has been proven, (2) a definite gain in strength and density or reduction in cost of materials were obtained, (3) dry mixes could be used that were difficult to spread by hand; reduction in water in some cases of half gallon per bag, (4) a slump of 1 in. may be used which gives higher flexural strength; however one half inch slump gives lower flexural strength, (5) concrete hardens more quickly, (6) spreading of dry concrete by machine is necessary, (7) that vibration is a construction process calling for coordinated plant and procedure, not merely the introduction of an additional unit of road building equipment.

New Jersey has used vibrated-concrete pavements longer than other states. In the paper on the New Jersey work, plant requirements are given. It was found to be difficult, costly, and unsatisfactory to spread some of the dry mixes by hand. A machine spreader is necessary to make use of the most economic dry mixes.

This very interesting and important method of using concrete can not be adequately treated in a review. Readers should find the above papers of much interest.

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Approximate design method for concrete skew rigid frames

EDWARD F. GIFFORD, (Asst. Engr., N. Y. C, R. R. Co.), *Engineering News Record*, May 3' 1934, p. 574. Reviewed by N. H. ROY.

OUTLINES a simple procedure for design of a skew frame by determining the relation between the span perpendicular to the abutments and the span parallel to the spandrel walls, calculating moments and thrusts per unit of width along the arch axis for the span perpendicular to the abutments, and then multiplying the moments and thrusts by $\sec^2\Theta$ (Θ being the angle between a plane parallel to the abutment and a plane perpendicular to the spandrel wall). The amount of steel is then determined as in the case of a right arch.

General recommendations as to types of design are made by the author upon the basis of skew-arch bridges that have been designed and built and are briefly: (1) skews up to 15 deg., design as right arch using section and span parallel to the spandrel walls, (2) skews 15—25 deg., design as right arch or square span, using transformation described above, (3) skews 35—50 deg., design as skew bridge, using above method for trial section and Mr. Hodge's explanation of skew-arch theory to be found in Arthur G. Hayden's "Rigid-Frame Bridges," (4) skews over 50 deg. use simple plate girder spans.

Vibration of concrete in large masses

Questionnaire by *Science et Industrie*, Feb., 1934. Reviewed by P. H. BATES.

THE entire Feb. 1934, issue of *Science et Industrie* is given over to answers to a questionnaire on the vibration of concrete. These answers are in papers by about 18 contributors —a number of the larger French concrete contractors, officers of some of the larger Government construction groups in France, and heads of a number of laboratories. In addition, a bibliography of publications in French, English, and German is included. This number does not lend itself well to abstracting. Not that the papers are not of extreme value, but due solely to the fact that to do justice to the papers, the abstracts of necessity would be too long for presentation herewith. This number is called to the attention of those members who are interested in this very important subject of vibration of concrete on the job.

The design of reinforced concrete sections subjected to bending

KARL KUGI, *Zeitschrift des Osterr. Ingenieur —und Vereines*, No. 7-8, Feb. 23, 1934, p. 47. Reviewed by INGE LYSE.

A STUDY of the viewpoint sponsored by Dr. Emperger that "the function of the compressive strength of the concrete in a reinforced concrete beam is to insure the utilization of the yield-point strength of the reinforcing steel." Formulas are developed for the percentage of longitudinal reinforcement required for the full utilization of the yield-strength of the steel at time of failure for given ultimate strength of concrete and yield-point stress of steel. Formulas are also given for the compressive strength of concrete required for developing the yield-point stress in the reinforcement. The useful compressive strength of concrete in flexure is taken as 1⅓ times the strength of the control cube. The concrete is assumed to have strength within 10 per cent of the design strength for controlled construction. Since the strength of the beams is governed by the yield-point strength of the reinforcement, high grade steel may be used in small quantities or low grade steel in large quantities. When the yield-point stress of the steel is not reached at ultimate load, there is an excess of reinforcement and a consequent waste. If the strength of the concrete is not fully utilized at time of failure, the economic disadvantage is not very great. The balanced type of reinforced concrete beam, that is, a beam in which the steel is stressed to its yield point at the same time as the concrete reaches its ultimate compressive strength in flexure, is the most economic design.

Weathering tests on concrete

L. O. HANSON. *Thirteenth Annual Proceedings of Highway Research Board*. Reviweed by W. V. MCCOWN.

PRESENTS the results of tests at the Materials Laboratory of the University of Wisconsin to determine the effect of amount of mixing water and type of coarse aggregate on the durability of concrete as measured by alternate freezing and thawing. The coarse aggregate was also tested by freezing and thawing and by sodium sulfate.

The coarse aggregate represents the three principal limestones which underlie a large part of southern Wisconsin. Sample A was from the Trenton limestone, an argillaceous, thin bedded magnesium limestone which shows considerable disintegration on faces exposed for many years. Sample B was also Trenton formation but harder, more thickly bedded and less weathered than usual for Trenton. Sample C was from Lower Magnesian limestone which is somewhat irregular in character, frequently cherty, and

variable in grain size. Sample D was from Niagara limestone, which is dolomitic, of fine texture, uniform quality, hard with low percentage of wear.

The rock was crushed in the laboratory, maximum 1½ in. washed, and fines under No. 4 were washed. Janesville sand, 65 per cent quartz with dolomite the other principal constituent, was used as fine aggregate; fineness modulus about 3.00.

Concretes were designed for three water-cement ratios, 0.65, 0.85, and 1.10 by volume, each with two slumps, 2- and 7-in. Specimens were 4 x 6 x 18 in. beams and 6 x 12 in. cylinders. All were held in moist room not less than 28 days. At 28 days, all compression specimens and one-third of the beams were broken. At 56 days, one-third of the beams were taken from the moist room and frozen and thawed, one cycle per day for 60 cycles, then broken. Freezing temperatures varied from 10° F. when placed in freezer to —8° F. 16 hours later. Beams were half immersed in water while freezing and totally immersed in water at 60° F. for thawing. The final third of the beams was removed from moist room at 6 months age and frozen and thawed for 60 cycles.

Rock samples were frozen and thawed, immersed in water at all times. The rock was dried after 25, 50 and 100 cycles and sieved. Other samples were tested with a modified form of A. S. T. M. Tentative Method (C89-32T).

The paper summarizes the results of the tests, discusses the results and presents the data in several tables and charts. The following summary is given:

Concrete

a. The resistance of the concrete to freezing and thawing was found to decrease directly as the water-cement ratio increased.

b. The quality of the coarse aggregate as determined by any of the given accelerated weathering tests was directly reflected in the resistance to freezing and thawing of the concrete.

c. The resistance of the concrete to freezing and thawing was somewhat increased by an additional length of curing prior to the weathering test.

d. Sixty cycles of freezing and thawing with partial immersion of the type used proved a severe weathering test for concrete cured in a moist room.

e. Strength losses due to freezing and thawing were not always accurately indicated by loss in weight or condition as determined by visual inspection, although the last two measures agreed very well between themselves.

f. No definite relationship was found to exist between the absorptive properties and the resistance to freezing and thawing of the concretes tested.

Coarse Aggregate

a. Freezing and thawing tests and a slightly modified form of the A. S. T. M. Tentative Method of Test for Soundness of Coarse Aggregate both graded the rocks in the same relative order of durability or resistance to weathering. The sodium sulfate test differed from the A. S. T. M. method in that the rock was not oven dried before being placed the first time in the solution, and after washing the sodium sulfate from the rock after 5 cycles, the rock was dried in laboratory air for a week instead of being oven dried. Percentage loss for the four rocks, 5 cycles of sodium sulfate:

A = 15.2 per cent
B = 14.6 per cent
C = 2.2 per cent
D = 1.2 per cent

b. The reduction in particle size due to 25 and 100 cylces of freezing and thawing was roughly duplicated by that due respectively to 5 and 10 cycle runs of a slightly modified form of the A. S. T. M. Soundness Test.

c. Change in either "fineness index" or "percentage loss" served equally well as measures of the amount of disintegration caused by weathering tests.

d. Evidence was developed that a measure of weathering resistance is given by the percentage loss of the large size rock particles alone.

Discussion by A. T. Goldbeck

These tests indicate that the durability of the concrete was controlled largely by the water-cement ratio. The water-cement ratios used in the wet consistency, high slump concretes were almost identical with those in the dry consistency, low slump concretes which indicates that more cement paste must have been used in the wet

mixtures. Therefore a greater volume of water must have been used in the high slump mixtures. Since cement requires a comparatively small amount of water for hydration, there must have been left over a larger amount of free water in the case of the wet mixtures than in the case of the dry mixtures. This free water creates porosity, and higher porosity in the wet than in the dry mixtures. In all probability, this will explain why the wet mixtures disintegrated to a greater extent than the dry mixtures in the freezing tests, notwithstanding the fact that both mixtures had the same water-cement ratios. These results are quite in line with those obtained in the National Crushed Stone Association laboratory where it was found that free water is a somewhat better index of the resistance of concrete to weathering than is the water-cement ratio.

The expansion specimens designed for a six inch slump exhibited far greater expansion than those designed for a two inch slump. This again suggests that uncombined free water is a factor in the expansion obtained.

Mr. Hanson suggests using the percentage of loss of weight of the rock samples after a given number of cycles of freezing and thawing as a proper index of the weathering resistance of that rock, the same sieve being used in the determination of percentage of loss as in the preparation of the sample. A serious error may be introduced by this method since fragments of rock, essentially sound, may become spalled off and pass the sieve used in preparation of the sample. These fragments would be called unsound by this procedure where, as a matter of fact, they may be entirely sound. The A. S. T. M. specifications are defective in this particular method of determining percentage of loss after test and it is suggested that the specification could be made satisfactory through the use of a screen having an opening one-half the size of the opening of the minimum size screen used in preparation of the sample.

Effect of surface characteristics and chemical composition of the aggregates on the strength of concrete

R. Grun, *Die Betonstrasse*, No. 4, Apr. 1934, p. 51. Reviewed by Inge Lyse.

The investigation reported in this paper included four types of aggregates with smooth surface, three types with rough surface, and three types of aggregates with chemically active surfaces. Tests were also made on concrete having hardened concrete, cement clinker, and light weight puzzolan materials as aggregates. The tests showed that the aggregates with smooth surfaces produced concrete of low tensile strength, while aggregates with rough surface produced considerably higher tensile strength. The effect of the surface characteristics was of less importance for the compressive strength. The data for different ages of test and different types of curing are presented both in tabular and graphical forms.

Growth and movement in portland cement concrete

C. G. Lynam; 135-X pages with numerous figures; $3.50; Oxford University Press. Reviewed by P. H. Bates.

This is a most interesting little book and should, as the author hopes, be of value to the cement worker, specialist in concrete, student, and practicing engineer. Its nature can be best stated in the author's own words: "This volume records no original experiments and is devoted entirely to the examination of published results. . . . . Contemporary research is so highly specialized that few workers deal with more than one point; and hence, while research, as a whole, has shed a flood of light on a large number

of isolated groups of phenomena, it has not made much progress towards showing relationships of these groups. In the following pages an attempt is made to show this relationship, and through the study of the existing material to develop a conception of the constitution of concrete, which on account of the nature of the material, is even more important in the study of concrete than metallurgy is in that of steel."

The volume is divided into seven chapters: I. Introductory, II. Hardening of Portlant Cement, III. Shrinkage and Internal Stress, IV. Bond, V. Elasticity and Creep, VI. Ultimate Strength and Working Stresses, VII. Unsettled Points, and finally a list of references.

The reading of this book does not convince one that the author has been successful in attempting to develop a concept of the constitution of concrete. On the contrary, one is convinced that the author himself is frequently in doubt as to the proper weight to be placed upon available data and how to correlate what at times are contradictory or inconsistent results. Thus, after citing the American Concrete Institute column tests, carried out in Lehigh University and University of Illinois laboratories, as showing that the two groups report the per cent of load transferred to the steel as varying by as much as 60 per cent, he states: "It is *probable* that the available strength of either the concrete or the steel is much greater than usually assumed." Other citations of uncertainty on the part of the author could be made and it is never clear what his conception of the constitution of concrete really is, although he does state that his studies "lead the author to the conception of the continuous growth in concrete for many years after it is cast." This seems to be based upon the gel nature of the product of the hydration of the cement and it further appears that upon the properties of this gel and its deportment in the presence of humidity or mechanical stress is based the author's concept of shrinkage, creep, elasticity, stress, bond, ultimate strength, etc.

The author is not to be adversely criticized for his possible failure in not having accomplished his most strenuous task. Indeed, he is to be congratulated upon the very terse and rather unbiased manner in which he has covered highly controversial fields, and the reviewer takes great pleasure in recommending the volume to those for whom the author intended it. It is an excellent compilation of work done in the various fields indicated.

Laboratory tests of multiple span reinforced concrete arch bridges

WILBUR M. WILSON, Research Professor of Structural Engineering, University of Illinois. *Proc. Am. Soc. C. E.*, Apr., 1934 (Vol. 60, No. 4, Part 1) p. 485-515. Reviewed by H. J. GILKEY.

RECORD of tests of five 3-span reinforced concrete arches as follows:

1. Rib without deck.
2. Rib, spandrel columns, and deck elevated well above the rib at the crown; expansion joints in the deck over the piers only.
3. Same except expansion joints were located over the piers and also at intermediate joints near the one-third point of each span.
4. Same as No. 2 except deck so low as to be integral with rib at the crown.
5. Same as No. 4 except for added expansion joints as in No. 3.

Structures No. 1, 2, and 3 were tested at pier heights of 20 ft., 15 ft., and 10 ft. while structures No. 4 and 5 were tested at a pier height of 20 ft. only.

Spans were 27 ft. each c. to c. and the crown was 6 ft. 9 in. above the springing line. Concrete mixture was

1:3:3 of w/c 1.2 designed for 2200 p.s.i. at 28 days.

IMPORTANT CONCLUSIONS

Structure without a deck (No. 1).

1. Elastic theory gives reasonable values for moment, thrust and shear.

2. Considerable cracking of the arch rib does not greatly alter the position or magnitude of the thrust.

3. The maximum stress was increased about 13 per cent for the 20 ft. piers over that for fixed ends.

Structures with decks

4. The deck decreases the moment carried at the springing line and increases the moment carried at the crown.

5. Intermediate expansion joints reduce the stiffness and moment carrying capacity of the central part of the structure.

6. The maximum stresses were increased 21 and 36 per cent over those for arches with fixed ends, for the pier heights of 10 and 20 ft., respectively.

7. For all structures the concrete in the arch developed the same unit stress as the same concrete in the 6 by 12 in. control cylinders.

Collapse of a fill and construction of a two-way bridge-canal of reinforced concrete at Artaix

M. MAGNIEN, Engineer of Ponts et Chaussees *Les Annales des Ponts et Chaussees*, 1934, I, Jan.-Feb. Reviewed by B. MOREELL.

DESCRIBES the collapse of a barge canal originally constructed in 1830-36. The canal at the point of failure had masonry walls and a concrete floor slab, all constructed with hydraulic lime. It rested on a fill of argillaceous sand covered by a blanket of clay approximately 6 ft. thick. Leakage from the canal into the fill over a long period caused sliding of the fill material and collapse of the fill and the canal.

The canal was reconstructed in reinforced concrete and now consists of a reinforced concrete channel 14 meters wide with a depth of water of 2.4 m. There are 15 spans; 14 of 7 m. and 1 of 10.35 m. The canal is supported on reinforced concrete columns, the 5 columns of each bent being capped with a reinforced concrete girder. The columns are founded on a continuous concrete mat with five longitudinal girders connecting the column bents to resist the upward pressure of the earth. The floor of the canal is a concrete slab 18 cm. thick supported on five longitudinal concrete girders which frame into the column cap. The sides of the canal are concrete slabs 13 cm. thick with a vertical span of 2.40 m., being supported at the top by a horizontal girder and at the bottom by the outside beams of the floor system.

High early strength portland cement was used for the entire construction to expedite the work. The working stress was 855 p.s.i. for concrete having 6.25 sacks of cement per cubic yard of concrete and 1000 p.s.i. for concrete having 7.15 sacks of cement per cubic yard.

The longitudinal floor beams and the columns into which they frame were designed as continuous rigid frames, but to diminish the magnitude of the moments in the columns they were given a minimum height of 7 m. and the smallest possible moment of inertia. For this purpose columns approximately rectangular in shape, 22 x 14 in., were used with the 14-in. dimension in the direction of bending. The columns were reinforced with 1.4 per cent of longitudinal steel and by two overlapping spirals. The size of the spirals is not given. Full advantage was taken of the increase in ultimate strength afforded by the spirals, as recommended by Considere, and the working stress on the concrete of the columns was taken as 1560 p.s.i. This includes the effect of the spiral. The forms were removed when the concrete

showed a compressive strength of 2980 p.s.i.

Vibrated concrete was used throughout with a mixture as dry as possible, to reduce shrinkage effects.

The entire interior of the canal is covered with 3 cm. of "gunite," reinforced with light metallic mesh.

No expansion or contraction joints were provided except at the end abutments, where steel rollers were provided. It was assumed in the design that 30 per cent of the total shrinkage would occur prior to filling the canal with water and that this shrinkage would disappear when water was placed in the canal. A maximum temperature variation of 15° C. was assumed and this was later found to be approximately correct by actual measurements. Temperature movements resulted in no apparent cracking of the structure.

Report on the suitability of concrete pipes for highway drainage

HJALMAR GRANHOLM, DONOVAN WERNER, and STIG GIERTZ-HEDSTROM, *Betong* (Journal of the Swedish Concrete Institute), No. 1, 1934, p. 1-84. Reviewed by INGE LYSE.

THIS report has been prepared by a committee of experts at the request of the Swedish Highway Department. Practical experiences had shown that the concrete pipes in many localities gave excellent service while in other localities disintegrations occurred. The investigation into the causes of the disintegration was carried out with a thoroughness worthy of comment, both with respect to the study of available test data as well as the planning and execution of the committee's test program. The main causes of trouble with concrete pipes were overloading and disintegration due to aggressive waters. Lack of compressive strength of the concrete in the pipes contributed to the failure by overloading, and the committee proposes that greater strength be required. High quality concrete was also found to resist the action of active waters much better than did low quality concrete. Asphalt coatings were found to be the most effective protective surface coatings used in the tests.

Besides laboratory tests, a full sized test conduit was constructed, using different kinds of concrete pipes as well as different surface coatings.

An experimental road of cement bound macadam

E. M. FLEMING and A. A. ANDERSON, *Thirteenth Annual Proceedings of Highway Research Board.* Reviewed by W. V. McCOWN.

During the summer of 1933, an experimental road, 1,200 ft. long and 10 ft. wide, was built of cement bound macadam by the Portland Cement Association. There was a total of 81 sections 10 or 20 ft. long grouped into 21 projects and each project was designed to study one or more variables.

Three sizes of crushed limestone, three sizes of gravel and one size of slag were used. Grout was mixed in five different proportions with three different gradings of sand, (0-4), (0-8), (0-14). Another variable was the amount of water in the grout. One brand of portland cement was used in all grouts. A study was made of ways to prevent loss of stone into soft subgrades. Different methods of compaction were studied, including hand tamping, vibration, and rolling. Three types of transverse joints were used, board joints, poured, and premolded expansion joints.

Eighty-six beams and 284 cores were taken from the pavement to determine the effects of the different variables on the strength of the pavement.

Subgrade—The type and condition of subgrade affects the amount of coarse aggregate pushed into it by compaction. The loss may be overcome by rolling into the subgrade comparatively thin layers of coarse material.

Aggregates—The type of coarse aggregate had no material effect on the strength of the pavement. Limestone with a wear loss of eight per cent is satisfactory. The optimum sizes of coarse aggregate range between ¾ to 2 in. and 1½ to 2½ in.

Grout—The "flow cone" developed during the tests furnishes a practical method for measuring or controlling grout fluidities. Fluidity of grout is affected by the size and gradation of the sand, proportions of cement and sand, and amount of mixing water. For the same sand, richer mixes segregate less than the leaner mixes. A volume of grout averaging 17 per cent in excess of the volume of voids in the coarse aggregate of the fully grouted pavement is required to compensate for loss of grout and shrinkage in grout volume due to escaping water.

Compaction—The pavement can be satisfactorily compacted by rolling, vibration, or hand tamping. A 5.8 ton roller gives the greatest compaction and highest strength and results in the greatest loss of coarse aggregate in the subgrade. Vibration gives slightly less strength, materially less compaction except with gravel, and less loss of rock in the subgrade, than the 5.8 ton roller. The 3.3 ton roller gives less strength and compaction than the 5.8 ton roller. Hand tamping gives less strength and compaction than the 3.3 ton roller, but negligible subgrade loss. To be effective, final compaction must be applied at a suitable interval after grouting. Rolling crushed stone or slag before grouting reduces voids and saves grout.

Construction Methods — Sprinkling ahead of grouting aids penetration. Penetration is assured when grout enters *at the bottom* of inspection holes when the grout on the surface is one foot or more away. Excess free water ahead of the grout should be drained from the subgrade. The proper interval between grouting and final compaction extends to the time when there is no appreciable free surface water and the slab is partially stabilized by early hardening of the grout. A longitudinal tamping template following final compaction aids in securing good riding surface. Wood, poured or premolded transverse expansion joints are satisfactory.

Strengths—The pavement with 1:2 (by weight) grout, and medium (0-8) sand developed the following strengths:

Compaction By	Modulus of Rupture		Compressive Strength
	7 Days	28 Days	28 Days
5.8 ton Roller	580	696	4,114
Vibration	550	670	4,014
3.3 ton Roller	429	626	4,234
Hand	388	567	3,891

Quantities of Materials—The amount of coarse aggregate used depends on its type, size, condition of subgrade and method of compaction. The amount of cement and sand used depends on proportions of the mix, type, size and grading of coarse aggregate, and amount of initial compaction. The average absolute volume of coarse aggregate comprises from 54 to 64 per cent of the total volume of the fully grouted pavement.

Strength of materials—structures of masonry, concrete and reinforced concrete

G. Pigeaud, *Le Genie Civil*, Vol. 104, No. 12, p. 264-267 and No. 13, p. 289-293. Reviewed by R. L. Bertin.

The author's introductory statement is that the above named materials are essentially heterogeneous in character and that the significance of their mathematical analysis requires a knowledge of the general properties of heterogeneous materials, a treatise on which was presented in *Le Genie Civil* of February 10 and 17, 1933.

The field fabrication of masonry and concrete and their variation in composition, variable ages when placed in service, result in great uncertainties. To establish their ultimate characteristics requires constant experimentation under conditions simulating as nearly as possible those prevailing in the field. The interpretation of the information thereby obtained relative to the modulus of elasticity, crushing, tension, shear and bond resistance, shrinkage at various ages and under varying stresses, requires a mathematical appreciation of the internal behavior of such materials.

He proceeds to derive fundamental equations giving the relative stresses in steel and concrete of reinforced concrete members subjected to (1) pure compression, (2) pure tension, (3) flexure both pure and composite:

Pure Compression

Under this heading is given the equation:

$$i = \frac{F}{E_a S_a + E_b S_b}$$

in which i = the shortening of the specimen suitably measured.
F = a force acting uniformly on the specimen.
E_a and E_b are the elastic moduli of the steel and concrete respectively.
S_a and S_b are the areas of steel and concrete respectively.
E_b being the only unknown can be derived therefrom.

It is pointed out that two values of $m = \frac{E_a}{E_b}$ should be obtained, one for loads of short duration and the other for loads applied slowly and permanently maintained, the latter introducing the effect of flow.

Pure Tension

Tests on such specimen have more theoretical than practical value, but bring out the law of the elongation of concrete found by Considere and presented before the Academy of Science as early as 1898, and in *Genie Civil* of February 4, 11, 18 and 28, 1899. This, briefly, is that if a prism of plain concrete is subjected to tension, the stress strain relation is slightly curved from O stress to the maximum F_o and the deformation at failure $i_o = \frac{F_o}{E_b S_b}$.

If a similar prism is reinforced with steel bars, and subjected to a tensile force F_s, that part of F_s, F, carried by the steel can be determined from the elongation of the specimen and the known value E_a of the steel. The valance $(F_s - F)$ remains equal to F_o whatever the value of $i > i_o$.

The concrete seems to have acquired ductility and stretches beyond i_o without offering additional resistance beyond F_o. In certain cases, Considere demonstrated that the concrete does not crack until the elastic limit of the steel is reached.

Mesnager, in more recent tests, demonstrated that the rupture of the concrete occurs when the steel stress, whatever be its elastic limit, reaches 12Kg. per mm.[2].

This leads to the conclusion that when a reinforced concrete prism is subjected to a tensile force, depending on the magnitude of the force, three successive stages exist (1) the elastic stage, (2) the stretching or plastic stage, (3) the stage of rupture. For each stage equations are given which have to do with the elimination or limitation of cracks in reinforced concrete sections submitted to tension.

Flexure Both Pure and Composite

This case is dealt with—(1) disregarding all concrete areas in tension, (2) considering the tensile resistance offered by the concrete in conformity with the teachings of Considere and Mesnager.

Space does not permit an adequate review of the mathematical derivations, except to state that they deal with relation between external forces and internal resistance, location of neutral axis, studies of reduced sections both arbitrary and real, effects of shearing stresses, and bond resistance for cases where all the concrete in the tensile zone is disregarded and where tensile resistance of the concrete within the tensile zone is considered.

The author concludes that in computations involving deformations, the entire section should be considered rather than the fictitious reduced section, for, as long as the internal stresses do not cause the concrete at any section to pass out of the ductile stage, the concrete for the greater part of the structure remains within the elastic stage and the real deformations do not depart materially from those obtained by the Elastic Theory.

He warns, however, that for structures designed so that the concrete in the tensile zones is in the stage of rupture, the deformations are no longer determinable by any known method.

Progress report on the reaction of calcium chloride on portland cement

PAUL RAPP and LANSING S. WELLS, *Thirteenth Annual Proceedings of Highway Research Board.* Reviewed by W. V. MCCOWN.

The Calcium Chloride Association has maintained a fellowship at the National Bureau of Standards to study the reactions of calcium chloride with cements and their constituents and to obtain information on the effect of calcium chloride on concrete made with present day cements. The long-time tests have not been completed. This report is on the short-

time tests (1 to 90 days) and presents data on the effect of calcium chloride on the heat of hydration, setting time, strength, and consistency of a selected group of cements.

Eight portland cements, one high-early strength, and two white portland cements were included in the investigation. Certain heat studies are reported also on 60 experimental cements of varying composition. The apparatus used for measuring directly the heat of hydration was that developed at the Bureau of Standards. The cement (200 grams) and water (67.5 grams) were thoroughly mixed in a small tinned can by a high speed mixing device. Copper-constantin thermocouples were inserted into the mixture and the can was placed in an air thermostat maintained at 21 ∓ 1 Degree C. From the data obtained, the heat evolved at any given period was calculated. Contributions of individual compounds to the heat were calculated by the method of least squares after computing the percentages of the compounds in the cements by the method of Bogue and assuming a linear relationship between the compound composition and the heat evolved. The calcium chloride used in the tests was a commercial, flaked, hydrated product containing 77.1 per cent $CaCl_2$ and the amounts added to the cements are reported as percentage $CaCl_2$ by weight of cement.

The specimens for testing the effect of calcium chloride were two inch mortar cubes and six by twelve inch concrete cylinders. All factors were controlled as far as possible. In the mortar tests, 0, 0.5, 1, 1.5 and 2 per cents anhydrous calcium chloride were added in the gauging water. For the concrete, 0, 1, 1.5 and 2.25 per cents calcium chloride were added.

Since the tests have not been completed, the following conclusions are tentative:

1—Calcium chloride increases the rate of heat evolution of all cements included in the investigation.

2—Calcium chloride increases the total heat of hydration of low heat cements and may decrease the heat of hydration of high heat cements at 24 hours.

3—The chemical composition of the cement has an important effect upon both the rate and total amount of heat of hydration developed during the first 24 hours. The general effect of calcium chloride over the range in compositions studied was a tendency to level the heat of hydration to an average increase of about four calories per gram at 24 hours.

4—The times of set of 11 commercial cements were decreased by adding increasing amounts of calcium chloride. The greatest proportional decrease was obtained by the addition of two per cent commercial calcium chloride.

5—Additions of calcium chloride did not affect the soundness of the cements.

6—The addition of calcium chloride increased the strength of the concrete made from the cements tested at all ages to 90 days, beyond which results have not yet been obtained.

7—The flow of concrete was increased by the addition of calcium chloride.

Concrete pavement cured with cement spray

SEARCY B. SLACK, *Thirteenth Annual Proceedings of Highway Research Board.* Reviewed by W. V. McCOWN.

THE State Highway Board of Georgia has tried a method of curing concrete pavement by which the surface is sprayed, immediately after finishing, with a thin layer of neat cement paste. An agitator mixes the paste in a mixing tank and a small air compressor forces the paste through the hose and nozzle. About 45 pounds pressure was required. One sack of cement was used for each 100 square yards of surface sprayed.

For comparison with the cement spray curing, concrete was cured with wet burlap 24 hours, and with burlap 24 hours followed by wet earth. All test sections were constructed during a hot, dry week in June. Cores were drilled at 28 days of age and tested the same day. The averages were as follows:

Cured by burlap and wet earth,
8 cores, 3,388 p.s.i.

Cured by cement spray,
12 cores, 2,981 p.s.i.

Cured by burlap 24 hours,
10 cores, 2,716 p. s. i.

Measurements of electrical resistance were also made on the concrete cured by the three methods as an indication of the amount of moisture in the concrete which in turn indicates the efficiency of the curing method. The resistance tests gave good correlation with the strength tests.

The evolution of the strength of cement during the last twelve years

EDMOND MARCOTTE, *La Revue des Materiaux de Construction et de Travaux Publics*, No. 295, p. 97-101, Apr. 1934. Reviewed by P. H. BATES.

THE author discusses the changes in strength that he has noted in a number of brands of cement during the last 12 years. He cites both the tensile and compressive strength as made in the usual acceptance tests in France. The average results yearly are given for a number of years. However, to show more directly the effect, he has prepared an average for the three years 1919-1921 inclusive and for the three years 1931-1933 inclusive. He finds that the per cent increases for the latter period over the former period in tension for the 2, 7, 28 and 90-day test periods are, respectively, 700, 70, 55 and 15. The per cent increases in compression for the same periods were, respectively, 500, 250, 120 and 70.

It would seem that in France the same condition has developed in the cement industry as in the United States, namely, the manufacturers have devoted their efforts to materially increasing the strength of their product at the early ages.

INDEX

PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

VOLUME 30—1934

From JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, Vol. 5, Sept.-Oct., 1933 to May-June 1934.

This Index combines references to:

Original contributions to the JOURNAL OF THE AMERICAN CONCRETE INSTITUTE—papers, reports, discussions and their authors.

Current Reviews, published by the Institute, of significant contributions to the literature, originating elsewhere. Each index reference to a Review is so designated.

In general, important subjects are classified and indexed under approximately 30 main headings each one appearing in its proper alphabetical order in bold face capital letters—as for instance, ARCHITECTURAL DESIGN. "Surface treatment" and other subjects classified under this head, are indented.

Key words to important subjects appear, in alphabetical order in addition to the general classification—as for instance "Admixtures" and "Blast furnace slag," each referring to MATERIALS AND TESTS under which all allied references appear, indented. Authors' names (except in references to papers *Reviewed*) appear in proper alphabetical sequence with the subjects, with references to their contributions.

Specific data on Beams are so indexed by reference to "ENGINEERING DESIGN" or "TESTS OF MEMBERS AND COMPLETED STRUCTURES" thus avoiding an oversight by the searcher of important allied data.

The readiest use may be made of this index by gaining some familiarity with the thirty main classifications.

The searcher for information on design and proportioning of concrete mixtures will look for MIXTURES and refer to subheads: "Design" and "Proportions." Field control methods will be found under TESTS AND FIELD CONTROL.

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